AN EXPERIMENTAL INVESTIGATION OF THE LOAD CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BY MEANS OF STATIC LOAD TESTS

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AN EXPERIMENTAL INVESTIGATION OF THE LOAD CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BY MEANS OF STATIC LOAD TESTS.

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The undersigned hereby certify that they have read and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled

AN EXPERIMENTAL INVESTIGATION OF THE LOAD CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BY MEANS OF STATIC LOAD TESTS

Submitted by PETER KOZICKI, B. Sc. in partial fulfilment of the requirements for the degree of

MASTER OF SCIENCE

ABSTRACT

The purpose of this investigation was to determine the ultimate carrying capacity of drilled cast-in-place concrete piles in typical Edmonton soils. The investigation was divided into two parts, the first part was primarily to check the design capacity of one of the drilled cast-in-place concrete piles used in the foundation for the Proposed Chemistry and Physics Building. The second part consisted of attempting to check the theoretical values of skin friction and end-bearing based on laboratory test results against the values as determined by means of actual pile load tests. Additional tests were also made to check the theoretical capacity based on laboratory test results of a combined skin friction and end-bearing pile.

Based on the field test results, it was found that:

(a) the pile that is part of the foundation for the Chemistry and Physics Building appears to have a factor of safety somewhat greater than 3.

(b) the test results on the friction pile were considerably higher than expected, possibly due to the tangential adfreezing strength of the soil.

(c) test results of the end-bearing pile check with the theoretical analysis.

(d) in the case of the combined skin friction and end-bearing pile, it appears that the tangential adfreezing strength of the soil affected the test results to a certain extent.

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CHAPTER I

INTRODUCTION

HISTORY

Since prehistoric times piles and pile foundations have been in common use as a means of support wherever the ground appeared incapable of sustaining the pressure exerted by the footings. Previous to the 19th century there was little or no basis for design of a pile foundation. As timber was abundant and labor cheap, as many piles were driven as the ground would permit. Settlement caused no concern as the prevalent type of structure could withstand a considerable amount of unequal settlement without injury.

In the 19th century, when industrial development created a demand for heavy but inexpensive structures in locations underlain by soft ground, the cost of pile foundations became an item of consequence, and engineers were expected to specify no more piles than were necessary to provide adequate support for the buildings. Consideration was also given to the use of steel, concrete, and composite piles of various types, which in comparison with timber piles, would provide a more permanent type of foundation as well as being able to support larger loads per pile. In order to be able to specify the most economical type of pile foundation, an engineer had to have some knowledge of the ultimate load that an individual pile could carry. Efforts to obtain the necessary information at a minimum expenditure of money and labor led to theoretical speculations that resulted in an impressive assortment of dynamic pile formulas. However, the realization slowly grew that the dynamic pile formulas had inherent shortcomings, and it became more

and more customary to determine the allowable load per pile on all but the smallest jobs by making load tests on test piles.

For piles such as the "Drilled Cast-in-Place Concrete Piles," where a dynamic method is not used in constructing the pile, the method of determining the allowable load per pile is a subject of considerable controversy among engineers. This type of pile was first used in the 1920's and the method originally employed in constructing the pile was to drill holes with a hand auger and fill them with concrete. The hand augers were later modified to use mechanical methods to turn the augers. These methods have today been replaced by power machinery capable of drilling holes from 16 inches to 96 inches in diameter, and to depths of 60 feet or more. In highly plastic clay soils the holes will normally stay open without casing until filled with concrete.

THEORY OF PILE ACTION

A pile transfers load into the surrounding soil by either (a) friction along the embedded length of the pile, (b) point-bearing, or, (c) combination of point-bearing and friction. Piles may be classified roughly as "friction" or "end-bearing", according to the manner in which they develop support. In any case, the load must be carried ultimately by the soil layers around and below the points of the piles, and an accurate knowledge of the compressibility of these soil layers is of utmost importance in predicting the load the pile will support.

FRICTION PILES

Friction piles in cohesive soils develop their carrying capacity from tangential forces along the sides of the pile and it can be assumed

that very little of the load is carried by the point. These tangential forces produce shearing stresses in the soil mass. The limiting value of the tangential resistance is therefore determined by the shearing strength of the soil in the vicinity of the pile, and by the frictional resistance between the soil and the surface of the pile. The points of such piles may carry some load because the soil at the elevation of the pile point is under restraint due to the weight of the soil above.

Friction piles driven in sensitive clay soils have provoked many and often conflicting ideas as to the damage done in the vicinity of the pile by remolding during driving. At one time it was felt that the driving of a pile in sensitive clays permanently reduced the shearing strength of the soil surrounding the pile. This has proven to be incorrect in most cases and it has been found that the soil will regain its full undisturbed shearing strength even in extremely sensitive soils. A full discussion on remolding and driving is given in (1) and (2). *

Friction piles driven in loose granular material serve to densify the material and as a result are some times referred to as "Compaction Piles". These piles are some times used to increase the relative density of the sand thus increasing the bearing capacity of the sand.

POINT-BEARING PILES

If a pile penetrates a stratum of soft soil onto a dense incompressible stratum, the load is carried entirely by end-bearing. Possibly some of the load may initially be carried by skin friction, however, it will eventually all be carried by end-bearing, (3). In some instances if a pile passes through a very compressible soil, the pressure

* Numbers in parenthesis refer to references contained in the bibliography.

transferred to this soil by skin friction will gradually consolidate it. Consolidation of this soil will continue until all of the applied load is carried by the pile point and if the load assigned to the pile exceeds the point resistance, then considerable settling of the pile may occur.

COMBINED POINT-BEARING AND FRICTION

Between the two extreme cases of full point-bearing and full friction with no point-bearing, there is a variety of possible combinations of point-bearing and side resistance. It is difficult to determine how much load is carried by the point and how much by the sides. In addition, it is not known how the tangential forces are distributed along the length of the pile. The distribution of stresses surrounding the pile is also unknown. Various methods of mathematical analysis have been applied to these problems but without much success to the present time.

If the pile is tapered, then in the case of skin friction piles in sandy soils, the friction on the sides is increased as displacement of the soil along the length of the pile must take place before any pile movement occurs. This has the effect of increasing the carrying capacity of the tapered pile in comparison with that of a parallel-sided pile having the same superficial area in contact with the surrounding soil. With end-bearing piles the effect is negligible, and the increased upper diameter fulfils no useful purpose from this standpoint.

IMPORTANCE OF THE SOIL ACTION IN A PILE FOUNDATION

In evaluating any pile foundation it is necessary to evaluate the action of the particular soil type involved as well as the behavior of the pile itself. The properties of the common building materials are relatively well known, and the designer can rely upon their strength and performance. On the other hand the properties and behavior of the

different classes of soil are, at best, only imperfectly known. Even when continuous samples are taken in a drill hole, only representative tests can be run to determine the soil properties and furthermore they may vary over the site investigated. Thus the designer and builder of a pile foundation must guard against possible variations of the soil properties on any particular site.

The total resistance of the soil to the penetration of a pile consists of:

1. The resistance that the soil has to displacement.
2. The resistance the soil has to a reduction or increase in the volume of voids in the soil adjacent to the pile.
3. The resistance of the soil to movement at the pile soil contact.

This resistance depends on the character and density of the soil and on the pressure exerted against the sides of the pile. In loose granular soil vibration of the pile driving operation causes a densification of the sand accompanied by a large increase in friction. Also in a loose granular soil the displacement at the point is practically all by reduction in the volume of voids. In saturated cohesive soils practically no volume changes occur, thus as the pile is driven bodily displacement of the soil takes place.

SETTLEMENT

Failure load (4) of an individual pile is considered as that beyond which an increase in load produces a disproportionate increase in settlement. Since a disproportionate increase in settlement is caused by shear failure of the soil, this criteria must apply only to cohesive soils. In granular soils there may not be a load where a

disproportionate increase in settlement occurs, therefore the definition for failure in granular soils does not mean shear failure but rather consolidation. As a result, the criteria for pile failure in granular soils is that the allowable load shall in no case exceed the load which produces a net settlement of 0.01 inches per ton of gross load applied as determined by a pile load test.

Whatever the general character of a pile foundation may be, the ability to relate the settlement of an individual pile to that of a pile group is of prime importance. Many investigators have found it extremely difficult to relate the action of a single pile to a group particularly when piles are surrounded by a granular material. However as the purpose of this investigation was the action of single piles the action of pile groups will be limited to the following discussion.

The magnitude of the difference between the settlement of an individual pile and that of a group depends on the characteristics of the materials shown in the soil profile. In this connection one must distinguish among the following four principal cases:

(1) The points of the piles are embedded in a firm stratum, with the underlying material of equal or less compressibility.

(2) The points of the piles penetrate through a bed of sand or gravel with a layer of clay below the base of the piles.

(3) The piles are embedded in a stratum of loose sand which extends for a considerable depth below the pile.

(4) The piles are embedded in soft silt or clay (floating pile foundation).

In the case of piles embedded in a firm stratum, with underlying

material of equal or less compressibility, very little settlement will occur. As a rule settlement ceases within a few months after all the loads are applied, provided the piles do not carry loads considerably in excess of their allowable loads.

For piles penetrating through a bed of cohesionless material with a layer of clay below the base of the piles, two different possibilities need be considered. If the pressure exerted on the clay by the foundation does not exceed the preconsolidation pressure, the influence of the beds of clay on the settlement of the foundation is unimportant. On the other hand if the pressure exerted by the weight of the building is great enough, the gradual consolidation of the layers of clay located beneath the points of the piles will cause settlement. The settlement is likely to progress at a decreasing rate during a period of many years. If the clay is soft and the layers thick, the ultimate settlement can be very large, although the factor of safety of the soil against failure may be fully adequate. For a given spacing of the piles and a given load per pile, the settlement increases with increasing size of loaded area.

In the case of piles embedded in a stratum of loose sand which extends for a considerable depth below the piles, a distinction has to be made between driving piles and drilled cast-in-place piles. As mentioned previously, driving piles densifies the surrounding sand which results in an increased skin friction. The ultimate bearing capacity of driven piles in sand increases roughly with the square of the depth of penetration (1). Large-scale experiments (5) have shown that compaction caused by driving one pile influences the bearing capacity of any other

pile located within a distance equal at least to five times the diameter of the pile. As a consequence if only one pile in a group is loaded, its settlement under a given load will decrease as the number of piles increases. Nevertheless, if all the piles are loaded, the settlement of the group under a given load per pile increases with the number of piles. Cast-in-place piles formed by drilling in loose sand do not increase the relative density of the sand as a result there is no large increase in the ultimate capacity as compared to driven piles. The ultimate capacity is dependent on the actual condition of the sand and would not be increased by the installation of additional piles. The difference in settlement between a single pile and a group of piles, each under the same magnitude of load as the single pile, would roughly be the same as for the case of the driven piles. The difference would be that, under the same magnitude of loading, the driven piles would settle considerably less than the drilled cast-in-place piles.

In the case of piles being embedded in a soft stratum the pile foundation would serve the purpose of transferring load to a lower level. If the soil is fairly homogenous to great depth, the settlement of a pile foundation carrying a given load distributed over a given area decreases appreciably with increasing length of piles. However, should the construction of a pile foundation be in an area of newly placed fill, problems may arise due to negative skin friction acting on the piles. If the subsoil consists of loose sand or highly permeable and relatively incompressible soils, the effect of the fill on the piles can be disregarded. On the other hand, if the subsoil contains layers of

soft silt or clay, the presence of the fill considerably increases the load on the piles, and, as a consequence, also causes an increase in settlement. Before the piles are installed, the compressible strata gradually consolidate under the weight of the fill material. After the piles are installed it can no longer settle freely because its downward movement is resisted by skin friction between the fill and the piles. An imperceptible downward movement of the fill with respect to the piles is sufficient to transfer onto the piles the weight of the fill located within the cluster. If this load is greater than the point resistance of the pile, the settlement of the foundation will be excessive, regardless of what the ultimate bearing capacity a load test may indicate.

The problem of arriving at the bearing capacity of piles by means of laboratory test results is difficult and is practically impossible for the case of piles driven in sand. The most reliable procedure for determining the bearing capacity of an individual pile in sand is a static load test. This is not always practicable and other tests such as the standard tests or the cone penetration tests are used. With cohesive soils the static load test is the best test for determining the bearing capacity of an individual pile provided the long term settlement effect is taken into account. The standard penetration test or the cone penetration test is not reliable in cohesive soils and the reliability of laboratory strength test results depends on the uniformity of the soil conditions. This investigation was to serve the purpose of checking the design method using laboratory strength test results against the results of static load tests.

PILE LOAD TESTS

There have been innumerable arrangements (7) of apparatuses developed for making loading tests on piles. A great deal of flexibility in design and ingenuity has been applied to make tests with the greatest economy of time and the use of available equipment.

Test loads may be applied by (a) a direct load such as heavy weights or water tanks placed on a platform; (b) jacking against a loaded platform or against an existing structure; (c) the use of anchor piles.

Direct load can be applied by any convenient means such as pig-iron, earth, water tanks, or precast concrete blocks. The procurement of sufficient fixed load is sometimes difficult, and the removal of such full load for repeated loadings and releases on the same pile, which is usually desirable, is nearly impracticable. Water tanks may be arranged for draining and refilling fairly readily. There is danger from improperly fixed loads. Corner supports should be placed close under loaded platforms to catch the load should tilting occur because of shifting of the load or yielding of the soil. Jacking against fixed load on platforms is preferable to resting the load on the pile. The platforms always remain resting on cribbing, thus eliminating the danger of the platforms tilting.

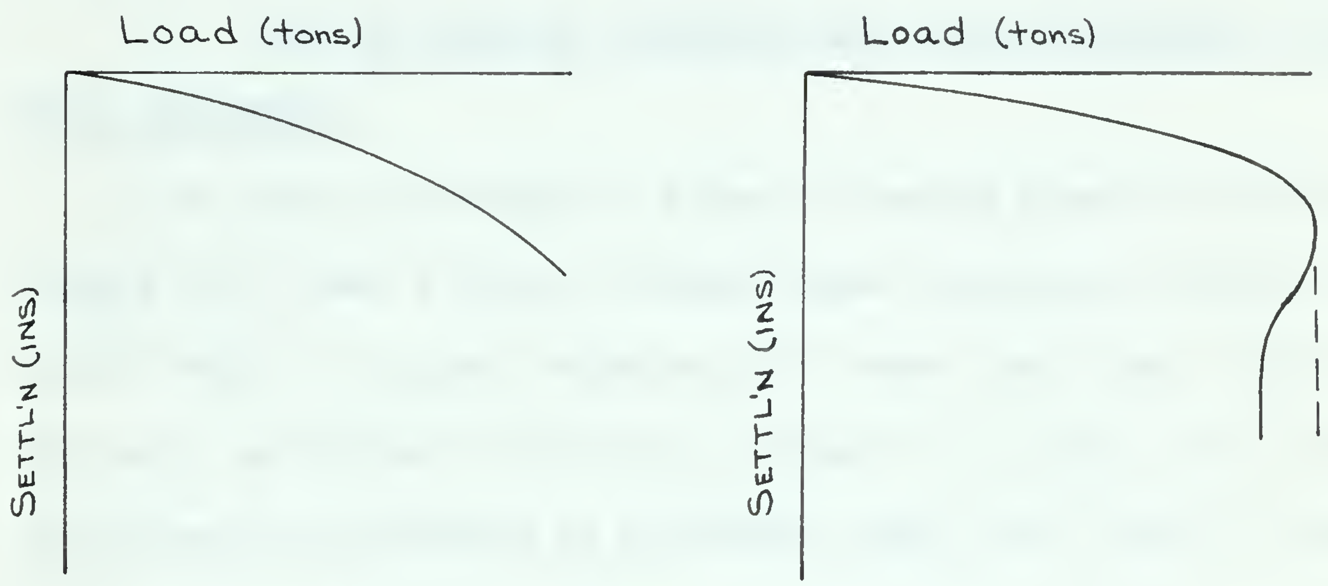
Jacking is usually done with hydraulic jacks employing a gas or liquid under pressure. Another advantage of jacking is that the loads can be applied and released quickly and at will, permitting quick determination of the net settlement of the pile, or movement in the soil, after rebound has occurred.

A compact set-up is obtained through the use of anchor piles as it does away with the cumbersome weights required for the fixed loads. Two anchor piles at least 5 feet on each side of the test pile may be used. Jacking is done against an I-beam which is fastened rigidly to the anchor piles. The anchor piles should be larger in diameter than the test pile and they should extend deeper into the ground in order to provide enough reaction to test the test pile to failure. Loads can be applied and removed simply by controlling the pressure on the jack.

Test loadings on piles are the final answer for determining the ultimate capacity of piles in sand. However for piles in clay, a short term test such as a pile load test is not entirely suitable as the cohesive strata cannot obtain its full settlement until after soil consolidation, which may take years. If the pile is in such a soil the ultimate compression cannot be determined, however the shearing value can be determined. Furthermore, the settlement of small loaded area of cohesive soil would have no relation to the much greater settlement which would occur under the same pile if it were one of many under the entire structure. It is impossible to evaluate tests unless adequate boring records present a complete picture of the soil conditions. Satisfactory pile load tests can be made in any soil through which water can seep freely in the voids.

Because of the difference in the permeability of clay and sandy soils, the Load vs Settlement Curves for the two types of soils have distinct characteristics. As shown below the curve for a pile loading

test in sand has a continuously increasing slope while in a test on clay the plot may be practically a straight line nearly to failure. After reaching its peak value, it will then drop back.



(a) Sand (b) Clay

PLOTS OF LOADING TESTS ON PILES

Therefore in the interpretation of test pile results it is highly important to have a complete understanding of the actual soil conditions as well as an appreciation of the group action of piles.

CHAPTER II

DESCRIPTION OF TEST PILES

Typical Sections of piles tested are shown on Plate 1 and are as follows:

(1) SITE OF PROPOSED CHEMISTRY AND PHYSICS BUILDING. Plate 1A.

SOIL CONDITIONS

The site is overlain by 6 feet of medium plastic stiff clay. From 6 to 15 feet a layer of medium dense non-plastic glacial till exists which is in turn underlain by a dense sand layer from 15 to 18 feet. At 18 feet the material changes to a sandy silt, low to non-plastic and contains an occasional pebble and layers of clay till. This material exists to a depth of approximately 25 feet and then changes to a dense glacial till.

TEST PILES

(a) Test Pile No. 1 - was one of the actual piles used for the foundation of the Chemistry and Physics Building. The pile was 12 inches in diameter, 23 feet long with a two foot diameter bell and was designed to carry 25 tons based on end-bearing only. The pile was reinforced with 4 No. 5 bars x 10 feet long. Since the pile formed part of the foundation for this building, the applied load was limited to twice the design load.

(b) Test Pile No. 2 - was constructed especially to be tested to failure. The pile was 12 inches in diameter, 23 feet long and was not belled. The reinforcing is the same as in Test Pile No. 1. Test Pile No. 2 was constructed four feet from Test Pile No. 1. It was assumed the soil conditions are identical for both piles.

(II) SITE OF WEST-END CITY YARD. Plate 1B.

SOIL CONDITIONS

The site is overlain from zero to 1 foot by a layer of gravel. From 1 foot to 21 feet below ground surface a deposit of firm highly plastic clay exists which is underlain from 21 feet to 22 feet by a layer of soft clayey silt. A deposit of firm to dense glacial till exists between 23 feet and 30 feet.

TEST PILES

(a) Test Pile No. 3 - was a 12 inch diameter pile, 26 feet long, and was not belled. No vertical reinforcing was used in the pile and it was to be loaded to failure.

(b) Test Pile No. 4 - was a 12 inch diameter pile, 23 feet long, no bell, and was suspended three feet above the bottom of the hole by means of a one inch thick steel plate which was held in position by four No. 5 rods. The rods were welded to the plate, and in order to support the plate in place while the concrete set up, the rods were welded to a five inch deep channel which in turn was supported at either end on 12 inch x 12 inch timbers. To prevent any concrete from passing through between the outside of the plate and the hole a 1/4 inch thick membrane, 14 inches in diameter was fastened to top of the 12 inch diameter steel plate, before lowering the plate down the hole. Test Pile No. 4 was constructed especially to determine the ultimate value of skin friction in a typical highly plastic Edmonton clay soil.

(c) Test Pile No. 5 - was a 12 inch diameter pile 26 feet deep, no bell. It was formed by using a 12 inch diameter steel casing

Photo 1 - Test Pile No. 5 - 12 inch diameter steel casing.

flared to 16 inches in diameter at one end as shown in Photo 1. Collars were welded at 8 foot centres on the outside of the steel casing to prevent any buckling which might occur during the testing of the pile. The outside diameter of the collars was 16 inches, which was the diameter of the drilled hole. The function of the flare at the bottom end was to give a confining effect to the soil just below the bottom of the pile. This pile was constructed to determine the effect of end-bearing, and was to be loaded to failure.

Since the time available to carry out this investigation was limited, 3,000 lb. high early strength concrete was used in all piles with the exception of Test Pile No. 1, where standard 3,000 lb. concrete was used.

All piles were tested in compression and no reinforcing was required in any of the piles with the exception of Test Pile No. 4, where the reinforcing was used to support the steel plate above the bottom of the hole.

The piles were drilled with a Hughes Williams Power Auger and the bell in Test Pile No. 1 was formed by means of a mechanical belling tool. Immediately after completion of drilling, concrete was poured into the hole, and the top of the pile was carefully formed in order to provide a level bearing surface.

CHAPTER III

TEST SETUP

It was felt that a pile setup carrying 70 ton of pig-iron kentledge would be suitable to test any of the piles to failure, if so desired. Economics was also a governing factor in limiting the test pile setup to 70 tons.

The test setup is shown in Photo 2 and Plate 2. A 175 ton hydraulic jack was used to transfer the dead load onto the pile. The amount of load transferred was recorded by means of the hydraulic jack which had been calibrated to 100 tons. Calibration test results are shown in Table 1 and the calibration curve is shown on Plate 3.

The pig-iron was supported on a 14 foot square platform and a 16 WF 36 I-beam was used to take up the load. The bottom of the I-beam was supported on a frame work of 12 inch x 12 inch timbers, four feet above ground surface. This provided sufficient room for placing of the jack on the top of the pile. A bearing plate 2 inches thick was used between the top of the pile and the jack, as shown on Plate 2. This bearing plate was necessary in order to prevent crushing of the concrete at the top of the pile.

A 5 inch channel supported at either end, was used as a reference point for the extensometer readings. For Test Pile No. 1, the supports used were one inch diameter rods, four feet long, driven two feet into the ground at a distance of 10 feet from the centre of the pile. This did not prove satisfactory as it was found that when over 37 tons of load was transferred to the pile, the extensometer readings did not agree with the level readings. If the load on the pile was less than

Photo 2 - Test Setup.

37 tons, then the two readings were in agreement. As a result it was felt that, as the load was being transferred from the timber crib to the pile, the ground tended to rebound. Since the ground was frozen at the time that the test was being performed, it is conceivable that this was occurring. In order to get around this problem it was decided to drive a two inch diameter pipe 4 feet into the ground. The rod used to support the channel was driven inside this pipe, Photo 3. This method proved satisfactory for fixing the channel for the balance of the testing program.

In order to check the extensometer readings a surveyor's level and target rod were used. The level was set up at approximately 50 feet from the test setup and a reference point at approximately the same distance away was used as a bench mark. In all cases with the exception of the test on Test Pile No. 1, the extensometer and level readings were in close agreement. The reason for the discrepancy in the readings on Test Pile No. 1 were discussed previously.

A magnetic holder as shown on Plate 2 and Photo 4, was fixed on the jack to hold the extensometer in place during the testing period.

Test Pile No's 1 and 2 were constructed at a distance of 4 feet apart as were Test Pile No's 4 and 5, in order that both piles could be tested with one test setup. In both cases shifting of a portion of the pig-iron kentledge was all that was required, to obtain the desired reaction load. This reduced the cost of the investigation as well as speeding up the testing.

Photo 3 - Channel Support.

Photo 4 - Setup Used to Record Settlement.

CHAPTER IV

ANALYSIS OF SOIL CONDITIONS

The holes for the Test Piles were all drilled by means of a power auger. Undisturbed soil cores were recovered during drilling by pressing 2-inch and 3-inch diameter thin walled shelby spoons into the bottom of each hole at regular intervals as drilling progressed. Soil cores recovered in this manner were carefully sealed with Paraffin and taken to the Civil Engineering Building at the University where they were subsequently analysed. In addition disturbed samples were taken at regular intervals for natural moisture content determination.

(1) SITE OF CHEMISTRY AND PHYSICS BUILDING

Detailed soil conditions together with field and laboratory test results for this site are given in Appendix A, and are summarized on Plate 2A.

No samples were taken in the drilling of Test Pile No. 1 and therefore since Test Pile No. 2 was only 4 feet away it was assumed that the test results of samples taken in the drilling of Test Pile No. 2 apply to both piles.

The drill log indicates that the site is overlain by 6 feet of medium plastic, stiff clay. From 6 to 15 feet a layer of medium dense non-plastic glacial till exists, which is in turn underlain by a dense sand layer from 15 to 18 feet. At 18 feet the material changes to a sandy silt, low to non-plastic and contains an occasional pebble and layers of clay till. This material exists to 23 feet below ground surface. No test holes were available near Test Piles 1 and 2 which

extend to a greater depth below ground surface, but from the author's experience on the site during installation of the piles, the dense silty material exists for a few feet deeper and then the material changes to a glacial till of equal if not greater density.

An analysis of the test results show that theclay has an unconfined compressive strength at 5 feet of 3.1 tons per square foot, at a moisture content of 25.0 percent of dry soil weight. The liquid limit is 40.6 percent and the clay has a plasticity index of 21.2 percent. These test results indicate that the clay is very stiff and of medium plasticity. The glacial till has an unconfined compressive strength of 4.4 tons per square foot, at a water content of 11.6 percent. This corresponds to a value of approximately 40 blows as obtained from the Standard Penetration Test in Test Hole 6 on the same material. No Standard Penetration Tests were run on the sand. The blow counts of two Standard Penetration Tests on the silt deposit in Test Hole No. 6 corresponding to a depth of 19 and 22 feet in Test Pile No. 2, was 100 in both cases. This indicates that the silt deposit is in a very dense condition.

An attempt was made to run triaxial compression tests on samples taken of the sand and silt deposits, but because of the density of the material, difficulty was experienced in extruding the samples out of the shelby tubes. As a result a suitable sample was not available on which a triaxial test could be performed.

During the drilling operation some caving of the sand layer was experienced but it was not too severe, and the cross section of the shaft is consistent throughout the entire length.

The subsoil conditions at the location of Test Piles 1 and 2 are very good but they should not be taken as representative for the entire site of The Chemistry and Physics Building. Over some of the other portions of the building site the conditions were not as suitable as in this particular location. The reasons for choosing the area near Test Pile No. 1 were:

(a) convenience

(b) accessibility

(c) pile carrying a small design load. As a result, a pile had to be chosen which best fitted the above conditions even though the subsoil conditions were better than the average.

(2) SITE OF WEST-END CITY YARDS

Detailed soil conditions together with field and laboratory test results for this site are given in Appendix B, and are summarized on Plate 2B.

The test hole log indicates a layer of gravel from zero to 1 foot, underlain by a firm highly plastic clay down to a depth of 21 feet. Between 21 and 22 feet a layer of soft clayey silt exists, underlain by a glacial till deposit. The glacial till is soft between 22 and 23 feet, but becomes dense at 25 feet. This deposit is of medium plasticity, grey in color with coal and pea gravel present. None of the test holes were deeper than 30 feet, but it is not possible, on geological grounds, for highly compressible material to exist below the depth investigated.

An analysis of the test results shows that the natural water content is consistent down to a depth of 22 feet, where it increases from

an average value of approximately 35 to 47.4 percent, and then drops off to 17.6 percent at 25 feet. The natural water content is near the plastic limit throughout the depth investigated with the exception of tests between 20 feet and 22 feet where it approaches the liquid limit, and then falls off to the same value as the plastic limit at 24 feet. When the natural water content in non sensitive clays approaches the plastic limit the soil is in a semi-solid condition, and when it approaches the liquid limit the soil is in a liquid condition.

The liquid limit tests indicate a very highly plastic clay at a depth of 5 feet with the liquid limit varying between 80 and 90 percent. The liquid limit tests on the clay down to a depth of 20 feet resulted in values greater than 50 percent. Because of the increase in silt content at 20 feet the liquid limit drops off to 42.8 percent. The plastic limit in all cases varies between 20 and 30 percent, therefore the plasticity index decreases with an increase in depth.

The Atterberg Limits on the glacial till indicate it to be a medium to highly plastic till, with the plasticity index in the neighborhood of 24 percent.

All the liquid limits on material from this site plotted above the "A" line on the plasticity chart indicating that the material is an inorganic clay. The band of curves formed by plotting the results of tests on the plasticity chart are shown on Plate 3B.

The average of 12 unconfined compressive strength tests on the clay material works out to 1.2 tons per square foot. This indicates that the clay is firm in respect to consistency. A plot of Moisture Content vs Log Compressive Strength of the clay material is shown on

Plate 4B. No definite trend is evident from this plot.

The average of three unconfined compressive strength tests on the glacial till is 1.7 tons per square foot with a maximum of 2.4 tons per square foot at 26 feet. It is likely that the maximum value would be a typical value for the glacial till since the material above 26 feet is affected by the wet silt deposit at 21 feet. This is brought out in the plot of Moisture Content vs Log Compressive Strength shown on Plate 5B. Even though the number of test results are few, there is a definite trend towards an increase in the unconfined compressive strength with a decrease in moisture content. All the holes made water at a depth of 21 feet below ground surface, because of the wet silty layer present at that depth. Because of this the holes were filled with concrete immediately after the drilling operation had been completed in order to prevent any softening of the material at the base of the pile. The amount of water that seeped in was small, but if the hole had been left open for an hour, probably four or five inches of water would have accumulated in the bottom of a 12 inch diameter hole.

CHAPTER V

TEST PROCEDURE AND ANALYSIS OF TEST RESULTS

TEST PROCEDURE

The procedure followed in conducting the pile loading tests is outlined in "Procedures for Testing Soils" published by the American Society for Testing Materials.

Test Pile No. 1 was not loaded to failure as it was one of the piles used to form part of the foundation for the Chemistry and Physics Building. The applied load was 50 tons or twice the design load, and was applied in increments of 25, 50, 75, 100, 125, 150 and 200 percent of the design load. For the balance of pile load tests, the 200 percent load increment was taken as the load provided by the Test Pile Set-up. Four readings were taken at 15 minute intervals for each increment of load. Extensometer readings were taken to the nearest ten thousandth of an inch and surveyor's level readings were taken to the nearest thousandth of a foot. After the 1 hour reading had been obtained for each increment of load, the next increment was applied until either the 200% load increment or pile failure had been reached. The maximum load was then kept on for a 24 hour period with readings taken at intervals. After the 24 hour period had expired the load was taken off to 50, 25 and zero percent of the full test load and the rebound was measured immediately after unloading. Rebound readings continued to be taken for at least 12 hours, after all the applied load had been removed. Results of load tests on all the Test Piles are given in Tables 2 to 9 inclusive.

ANALYSIS OF TEST RESULTS

(1) SITE OF CHEMISTRY AND PHYSICS BUILDING:

Test Pile No. 1 was a 12 inch diameter pile, 23 feet long with a two foot diameter bell and was designed to carry 25 tons.

The load vs Settlement Curve for Test Pile No. 1 is shown on Plate 3A and the results are given in Table 2.

(a) Pile Behaviour

The total applied load was 50 tons which was 200 percent of the design load. No pile failure occurred using the criteria that pile failure occurs when an increase in load produces a disproportionate increase in settlement (4). Using this criteria, which is put forth in the National Building Code (1953), the allowable design load permitted on this pile is greater than 25 tons. The design load permitted should in no case exceed the load which produces a net settlement of 0.01 inches per ton of gross load applied. The net settlement at 50 tons of applied load was 0.06 inches and the allowable settlement permitted by the Code at an applied load of 50 tons is 0.50 inches. Therefore on the basis of either criteria no pile failure occurred and it would appear that the pile has a factor of safety greater than 3.

(b) Theoretical Analysis

The pile is embedded from zero to 6 feet in clay, from 6 to 15 feet in glacial till, from 15 to 18 feet in dense sand and from 18 to 23 feet in a sandy silt. The shear strength of the clay is 1.5 tons per square foot, and that of the glacial till is 2.2 tons per square foot. For the clay and the glacial till the shearing resistance is taken as one-half the unconfined compressive strength, but for the case of the granular

material from 15 to 23 feet it is difficult to compute the shearing resistance of the soil along the sides of the pile. However, assuming that the shearing resistance for the granular material along the sides of the pile can be determined from the following equation:

$s = p \tan \phi$ (5), where s = shear strength of the material, p = effective or intergranular pressure, $\tan \phi$ = coefficient of internal friction, ϕ = angle of internal friction.

The angle of internal friction for a dense uniform sand that consists primarily of rounded grains is approximately 35°. Assuming an average angle of internal friction in this case for the sand and silt material between 15 and 23 feet as equal to 25°, and an average effective pressure of 1 ton per square foot, the average shear strength of this material works out to 0.47 tons per square foot. The total ultimate value for skin friction on this pile would be the sum of the skin friction of each of the materials penetrated. This works out to 28 tons for the 6 feet of clay penetrated, 62 tons for the 9 feet of glacial till penetrated, and 12 tons for the 8 feet of sand and silt penetrated, making a total of 102 tons for skin friction.

To determine the end-bearing capacity of a two foot diameter bell the formula $q_{dr} = \gamma D_f N_q + 0.6 \gamma r N_\gamma$ (5), was used. In this formula q_{dr} = ultimate bearing capacity of a circular footing. γ = density of the soil at the base of the pile. D_f = depth of base of pile below ground surface. N_q = bearing capacity factor depending on the value of ϕ. r = radius of circular footing. N_γ = bearing capacity factor depending on ϕ.

Using a value of ϕ = 25° in determining the bearing capacity

factors, and assuming that the density of the soil at 23 feet is 130 lbs. per cubic foot, then the ultimate bearing capacity of a two diameter bell is 20 tons per square foot. Therefore the ultimate capacity of this pile based on end-bearing only should be 62 tons. Combining the calculated ultimate capacity for skin friction and end-bearing the calculated ultimate capacity of this pile should be 164 tons.

In order for peak values to be used as calculated above the stress strain characteristics must be the same for all the material through which the pile has penetrated. It is extremely unlikely that it would be and therefore peak values would not be fully developed. Therefore the maximum calculated capacity of the pile would not likely be the true maximum. However, the theoretical analysis does tend to confirm the conclusion based on the result of the load test that the pile probably has a factor of safety somewhat in excess of 3.

Test Pile No. 2 was a 12 inch diameter pile 23 feet long with no bell and was constructed to be loaded to failure.

The Load vs Settlement Curve is shown on Plate 3A, and the results are given in Table 3.

(a) Pile Behaviour

No pile failure occurred at the total applied load of 70 tons, based on criteria previously mentioned. The net allowable settlement under an applied load of 70 tons is 0.70 inches and the net settlement recorded under the applied load was 0.2729 inches. As a result there must be a considerable effect due to skin friction.

The total settlement recorded was 0.3617 inches of which 28 per

cent or 0.0988 inches is due to elastic deformation.

(b) Theoretical analysis

The pile penetrates the same material as in the case of Test Pile No. 1, and since it is also the same diameter, the ultimate calculated value for skin friction of 102 tons must also apply to this pile. The base of the pile in this case is 1 foot in diameter and the ultimate capacity based only on end-bearing should be 16 tons. Then theoretically the failure load of this pile should be 118 tons. Here again, the calculated ultimate values would have to be considered as peak values since the stress strain characteristics for the soils penetrate are not the same.

The Load vs Settlement Curves indicate that Test Pile No. 2 settled 35 percent more than Test Pile No. 1 at an applied load of 50 tons. This is due to the smaller bearing area of Pile No. 2. However most of the difference in settlement took place for the loads up to 25 tons, and beyond that the curves are parallel. It thus appears that the only function of the larger bearing area in this case is to reduce the total settlement of the pile. The difference in total settlement of the two piles at the design load of Test Pile No. 1 is 0.05 inches. Considering that at 25 tons applied load the total settlement of Test Pile No. 2 was 0.085 inches, it would appear that for the type of structure proposed, a 12 inch diameter friction pile 23 feet deep would have been suitable in this case.

(II) SITE OF WEST-END CITY YARDS

Since Test Pile No. 3 is a combination of end-bearing and skin friction the test results will not be discussed until the test results

of Test Piles 4 and 5 have been analysed.

Test Pile No. 4 was a 12 inch diameter pile, 23 feet long, with no bell and was suspended 3 feet above the bottom of the hole. This pile was constructed to determine the effect of skin friction.

Two tests were performed on the pile and the Load vs Settlement Curves are shown on Plate 6B, and the Test Results are given in Tables 4 and 5. In Trial No. 1, the pile was loaded to 37.5 tons and then as there was insufficient load on the test pile setup to apply the next increment of load, the pile was unloaded for 24 hours. The test was then resumed and the pile loaded to failure. This possibly is the explanation for the plot of the Load vs Settlement Curve for Trial No. 1 not forming a smooth curve.

(a) Pile Behaviour

Pile failure occurred at an applied load of 65 tons based on the criteria of disproportionate settlement, and therefore an allowable design load of 30 tons would be permitted on this pile. The net settlement at 30 tons applied load was approximately 0.08 inches while on the bases of the second criteria a net settlement of 0.30 inches would be permitted. The net settlement of this pile after 24 hours under an applied load of 65 tons was 2.146 inches for Trial No. 1 and 2.164 inches for Trial No. 2.

The elastic deformation in Trial No. was 2.3 percent or 0.05 inches. In Trial No. 2 the elastic deformation was 3.6 percent or 0.08 inches.

(b) Theoretical Analysis

The highly plastic clay material on this site has a Liquid Limit varying between 52 and 89 percent and the average moisture content is

approximately 36 percent. Therefore the clay should behave as a saturated cohesive material.

The pile is embedded in 1 foot of gravelly material which is underlain by a highly plastic clay layer from 1 to 21 feet below ground surface. Below this exists a 1 foot thick stratum of sandy silt which is in turn underlain from 22 to 26 feet by glacial till. The clay is stiff and on the basis of 12 test results has an average shear strength of 1200 lbs. per square foot. No tests were run on the sandy silt layer which is low to medium plastic and has a very soft consistency. The unconfined compression strength of the glacial till increases from 1.2 tons per square foot at 23 feet to 2.4 tons per square foot at 26 feet. The top of the glacial till has softened up due to the presence of the wet silt layer above it. Based on an average of three tests the shearing strength of the glacial till is 1600 lbs. per square foot. The ultimate capacity of this pile should then be the sum of the skin friction for the length of pile penetrating the clay, the length penetrating the silt and the length penetrating the glacial till. It is assumed that the shear strength of the gravel fill is equal to the average of the clay. It will be further assumed that the shear strength of the silt deposit is equal to 0.12 tons per square foot (13). Also, since the pile penetrates only into the soft glacial till, therefore in calculating the skin friction for that portion of pile, a shearing strength equal to 1200 lbs. per square foot will be used. This is based on the sample tested at 23 feet. Using the above values, the average shearing resistance of the soil is 1160 lbs. per square foot, and the theoretical ultimate capacity of the pile works out to 42 tons. This is considerably less than the load of

65 tons which was required to fail this pile. Since we are dealing with a saturated non-sensitive clay soil the calculated ultimate capacity should not vary greatly from the ultimate capacity as determined by means of a pile load test. The difference between the two results is considerable in this case, therefore possibly the top 3 feet of the pile which penetrated frozen soil affected the test results.

In arriving at the actual value for the shear strength along the length of the pile based on the failure load, the weight of the pile should be added to the applied load of 65 tons. The weight of a column of concrete 12 inches in diameter and 23 feet long is 1.35 tons, therefore the failure load should be taken as 66 tons. On this basis the average ultimate shear strength of the soil works out to 1800 lbs. per square foot for a pile 23 feet long. This is 640 lbs. per foot higher than the value for the shear strength of the soil based on laboratory test results. Assuming that the top 3 feet of frozen soil accounted for the difference of 24 tons between the calculated and the actual ultimate capacity of the pile, then the tangential adfreezing (6) strength of this material works out to 5,000 lbs. per square foot.

The tangential adfreezing strength is defined as the resistance to the force that is required to shear off an object which is frozen to the ground, and to overcome the friction along the plane of its contact with the ground. The value of the tangential adfreezing strength varies with many factors (20). In comparison the value given for a clay soil at a temperature of + 12^{0}F, and at a moisture content of 34.6 percent

is 24.7 tons per square foot. Assuming that the full value of the tangential adfreezing effect is developed in the top 3 feet of frozen soil, then an applied load of 242 tons would be required to overcome this effect. Since pile failure did occur, and the theoretical analysis does not agree with the actual field test, it would have to be assumed that the freezing-in of the top 3 feet had some effect on the test results, but to a considerably smaller extent than indicated by the above analysis.

Two tests were run on this pile in an attempt to determine the effect of remolding of the clay after shear failure had occurred and also to try and determine the effect of freezing-in of the pile on the ultimate bearing capacity. The test results did not vary sufficiently to draw any conclusions of the above effect on the ultimate capacity.

Test Pile No. 5 was a 12 inch diameter pile, 26 feet long, and flared out to 16 inches in diameter at the bottom end (Photo 1). The height of the flare was 8 inches. Test Pile No. 5 was bearing on glacial till and was constructed to determine the effect of end bearing in a pile foundation.

Load vs Settlement Curves for the three tests performed on this pile are shown on Plate 7B, and the results of the three tests made on this pile are given in Tables 6, 7 and 8.

(a) Pile Behaviour and Theoretical Analysis

Trial No. 1: based on the criteria of disproportionate settlement, pile failure occurred with the application of the first test load increment of 12.5 tons. The net settlement under this load was 1.56 inches therefore on the basis of the second criteria pile failure also

occurred as the actual net settlement is considerably greater than the allowable of 0.12 inches. The ultimate bearing capacity of the glacial till at 25 feet for a circular footing is 8.9 tons per square foot and allowing 1.3 tons per square foot for the confining action of the soil at 26 feet, the theoretical ultimate bearing capacity of this material would then be 10.2 tons per square foot. On this basis the theoretical ultimate capacity of a 16 inch diameter pile bearing at 26 feet below ground surface should be 14 tons which agrees with the ultimate capacity as determined by the pile load test.

Loading of the pile was continued until a load of 50 tons had been applied, resulting in a net settlement of 7.416 inches. The load was then removed and the pile was immediately reloaded.

Trial No. 2: based on the first criteria as stated above, pile failure did not occur until the 50 ton load had been applied. On the basis of the second criteria the pile failed at an applied load of 37.5 tons as the net settlement permitted is 0.375 inches which is considerably less than the resultant settlement of 0.6357 inches. On the basis of 37.5 tons being the failure load, the ultimate bearing capacity of the glacial till works out to 27 tons per square foot which is considerably greater than the ultimate on the basis of unconfined compressive strength results. Since the glacial till is a heterogeneous material the unconfined compressive strength test does not give a true indication of the shear strength of this material. Also because no denser layer exists immediately below the pile, as indicated by the strength test on the sample at 30 feet, the glacial till material must offer a considerable greater resistance to penetration than is indicated by the

unconfined compressive strength test. A better indication of the penetration resistance of the glacial till would have been obtained from a standard penetration test, but as no equipment was available to run this test, it was decided to run unconfined compression tests instead.

The net settlement of the pile 24 hours after the 50 ton load was applied was 6.35 inches, and thus the total settlement for the first two tests was 13.76 inches. The elastic deformation was 2.7% or 0.1738 inches.

Trial No. 3: based on either of the two criteria for pile failure, no pile failure occurred at an applied load of 55 tons. The net settlement 24 hours after the 55 ton load was applied was 0.384 inches as compared to the allowable of 0.55 inches. The elastic deformation was 47.6 percent or 0.348 inches. Even though possibly some of the load is being taken by skin friction on the flared out portion of the pile, it is difficult to make any analysis of this, and since the flared out portion is only 8 inches high, the amount of skin friction would be small in comparison to the end-bearing. Assuming that penetration is being resisted by end-bearing alone then at 55 tons applied load, the bearing capacity being developed is 40 tons per square foot. It would appear that on the basis of the three test results, the resistance of the glacial till to penetration increases considerably with a relatively small penetration of the test pile.

On the basis of the three tests performed on this pile it would have to be assumed that the unconfined compression strength test results together with the necessary adjustment for the depth factor give

accurate results for the end-bearing capacity of a drilled cast-in-place pile. In the case of a pile being driven into the glacial till and therefore causing displacement of the material, the standard penetration test would be more suitable for determining the design load.

Test Pile No. 3 was a 12 inch diameter pile, 26 feet long with no bell, and was constructed to be loaded to failure.

The Load vs Settlement Curve is shown on Plate 8B, and the test results are given in Table 9.

(a) Pile Behaviour

No pile failure occurred at the applied load of 70 tons. This is in accordance with the criteria of disproportionate settlement. Also, on the basis of the second criteria no pile failure occurred as the allowable net settlement permitted is 0.70 inches and the actual net settlement that occurred was 0.2221 inches. The elastic deformation was 25.8 percent or 0.0774 inches.

(b) Theoretical Analysis

Theoretically this pile derives its carrying capacity from a combination of skin friction and end-bearing, as compared to the simple case of a friction pile in Test Pile No. 4 and an end-bearing pile in Test Pile No. 5. The ultimate theoretical value for skin friction is the sum of the skin friction through 21 feet of clay, one foot of silt and 4 feet of glacial till. It will be assumed that the 1 foot of fill to have a shear strength equal to the average shear strength of the clay. The theoretical ultimate value for skin friction through the fill and the clay, based on an average shear strength of 1200 lbs. per square foot

is 40 tons. Here again assuming that the one foot silt layer has an average shear strength of 0.12 tons per square foot, then the theoretical ultimate value for skin friction through this layer is 0.4 tons. The theoretical ultimate value for skin friction through the glacial till, based on an average shear strength of 1600 lbs. per square foot is 10 tons. Therefore the theoretical ultimate value for skin friction on this pile is 50 tons. Allowing 1.3 tons per square foot for the depth factor then, the theoretical ultimate end-bearing capacity of a 12 inch diameter pile bearing on the glacial till at 26 feet is 8 tons. Combining the ultimate values for skin friction and end-bearing, the pile should theoretically have an ultimate capacity of 58 tons, assuming the clay and the glacial till to have the same stress strain characteristics.

Since the pile was not tested to failure and, as in the case of Test Pile No. 4, freezing-in of the pile did occur, it is difficult to draw any conclusions from the theoretical analysis based on the field test results. As in the case of the friction pile it is likely that the freezing-in of the pile affected the test results to a certain extent.

For comparison purposes a plot of Load vs Settlement Curves for all the Test Piles with the exception of Test Pile No. 5 is shown on Plate 7B. In comparison of 1 hour readings the least settlement was recorded in Test Pile No. 1, up to a test load of 36 tons. For test loads between 36 and 70 tons, Test Pile No. 3 recorded the least settlement. The Load vs Settlement Curve for Test Pile No. 2 is below that for Test Pile No. 3 and the difference increases with an increase

in load. The difference at 70 tons was 17 percent while at 50 tons it was 14 percent. This does not agree on the basis of strength test results for the two types of soils but does agree as far as the theory of consolidation for rate of settlement is concerned. In the case of Test Pile No. 3 because of impervious nature of the clay as compared to the sand, consolidation does not take place until a considerable time after the load is applied. In comparing the load settlement curves for Test Piles No. 1 and 3, the increased end area in Test Pile No. 1 had no effect in reducing the settlement for loads greater than 36 tons.

Curves for Test Pile No. 4 are below those of Test Piles 1, 2 and 3, with the exception of that between 21 and 45 tons, where the curve for Trial No. 2 is above that of Test Pile No. 2. This is possibly due to remolding effect of the clay soil.

Based on 24 hour readings the settlement of Test Pile No. 2 was 0.1237 inches while Test Pile No. 3 settled 0.216 inches, which is in agreement with the theory of consolidation.

A plot of stress vs strain based on field test results for the skin friction and end-bearing piles is shown on Plate 10B. The modulus of elasticity for the soil along the sides of the pile is greater than for the soil at the bottom of the pile. The plot of the two Trials for the friction pile are in close agreement. In the case of the end-bearing pile, Trial No. 3 definitely has a higher modulus of elasticity than Trial No. 2. This plot indicates that the material along the sides of the pile is stiffer than the material at the base of the pile. As a result in the case of the combined skin friction and end-bearing pile, the skin friction component would carry the greater percentage of the applied load.

A summary of pile load test results is given in the following table:

TABLE A - SUMMARY OF TEST RESULTS

ITEM	PILE NO. 1	2	3	4	5
Applied Load (tons)	50	70	70	65	55
Measured Ultimate Load (tons)	-	-	-	65	12.5
Ultimate Load Based on shear strength of the soil (tons)	164 (-)	118 (-)	58	42	14

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS:

The following conclusions can be drawn from the results obtained from the testing program.

(A) Site of Chemistry and Physics Building.

(1) Piles tested did not fail under test load. It would appear that the pile tested forming part of the foundation for the Chemistry and Physics Building has a factor of safety somewhat greater than 3. This was expected as the laboratory strength test results indicated that the sub surface conditions at this particular location were better than at other locations on this site.

(2) Since neither of the piles was loaded to failure, it is difficult to draw any conclusions on the accuracy of the theoretical analysis.

(B) West-End City Yards.

(3) Test Results for the friction pile did not check with theoretical analysis and the most reasonable explanation appears to be due to the fact that the top 3 feet of pile was in frozen soil.

(4) Test results for the end-bearing pile check with the theoretical analysis.

(5) Test results for the combined skin friction and end-bearing pile do not check with theoretical analysis, and here again the discrepancy appears to be due to the effect of the frozen soil.

RECOMMENDATIONS:

For possible future research into the carrying capacity of cast-in-place piles the following points are put forward:

(1) The test set up was satisfactory for the tests performed. In any future testing it may be found that the testing could be performed more economically by the use of anchor piles.

(2) For better anchorage of the reference points, a hole down to a depth of about 10 feet, filled with concrete could be used for short term tests, and down to at least 20 feet for long term tests.

(3) A nest of springs may be used on the top of pile in order to better maintain each increment of load especially when a constant load is to be maintained for any length of time.

(4) There was no effort made to determine the variation in bearing capacity of cast-in-place concrete piles with time. It had been hoped that conclusions could be drawn on the amount of skin friction that could be considered in the design of cast-in-place pile foundations, but since the modulus of deformation was the same for both types of soils therefore no conclusions can be drawn. It is hoped though, that enough data has been presented to stimulate interest in cast-in-place piles sufficiently to warrant more research in order to establish a uniform design criteria for foundations of this type.

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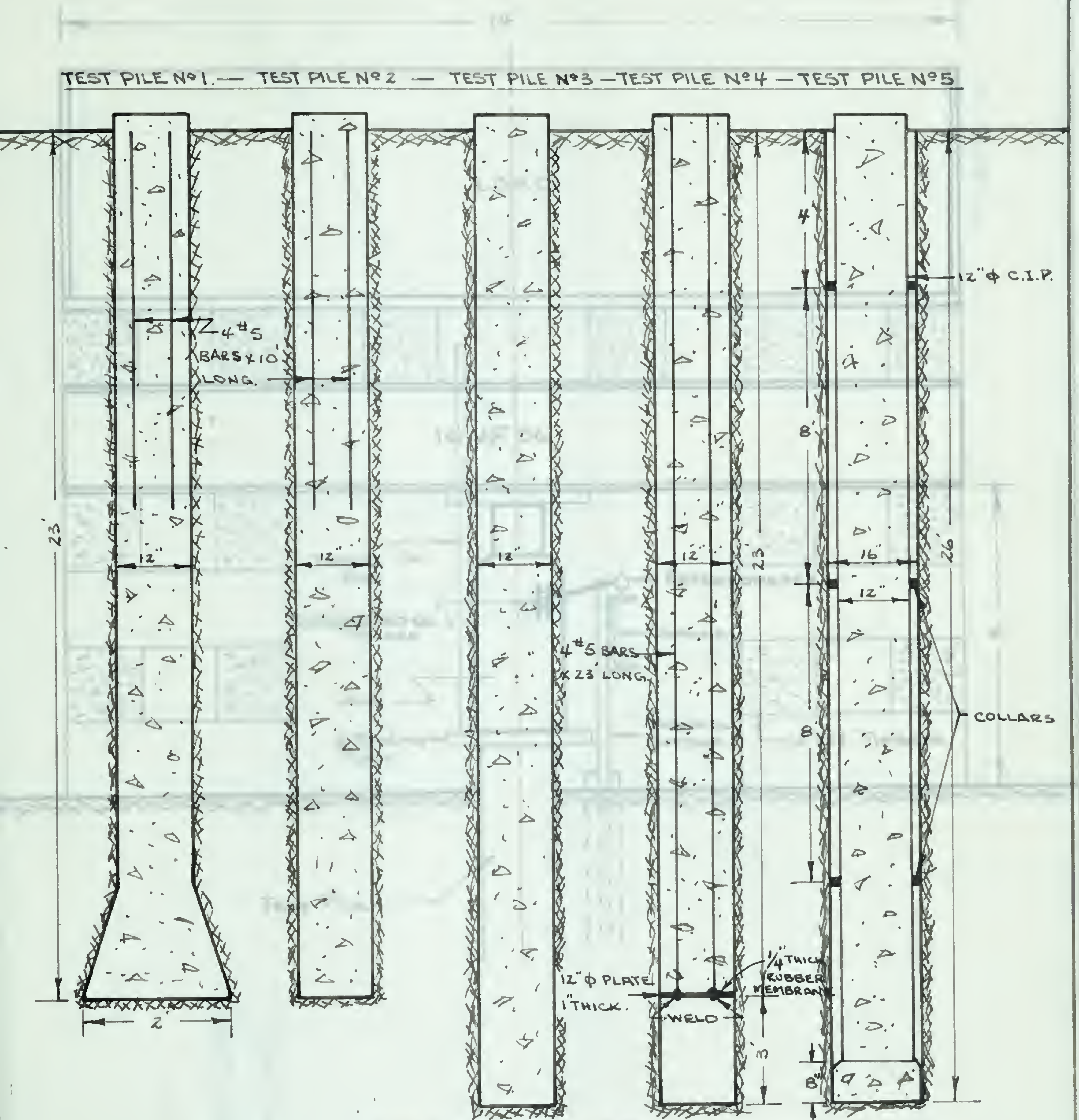
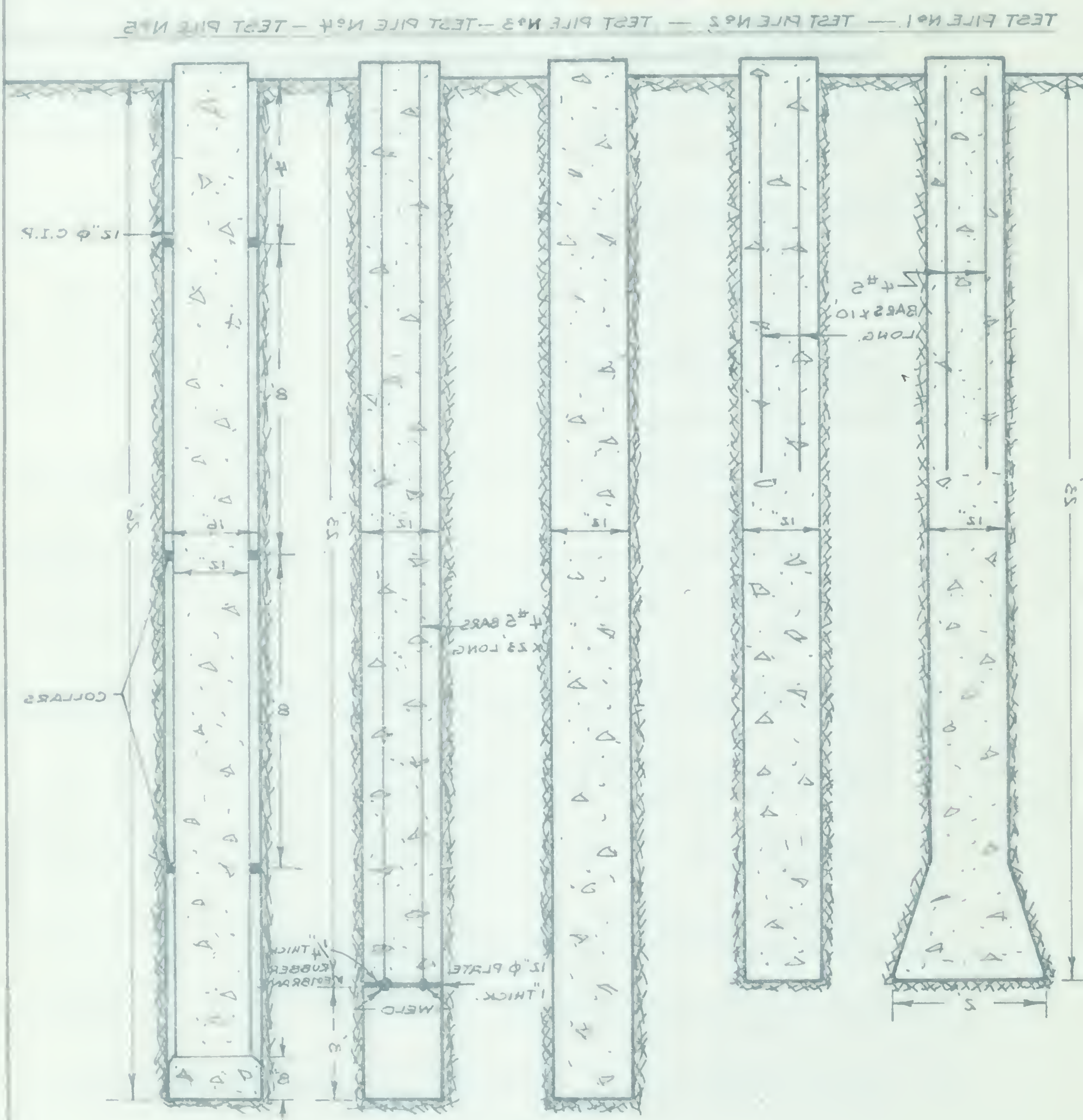


TYPICAL SECTIONS OF PILES TESTED

SCALES - HORIZONTAL 1" = 2'
VERTICAL 1" = 4'.

FEBRUARY 26, 1959.

PLATE I.

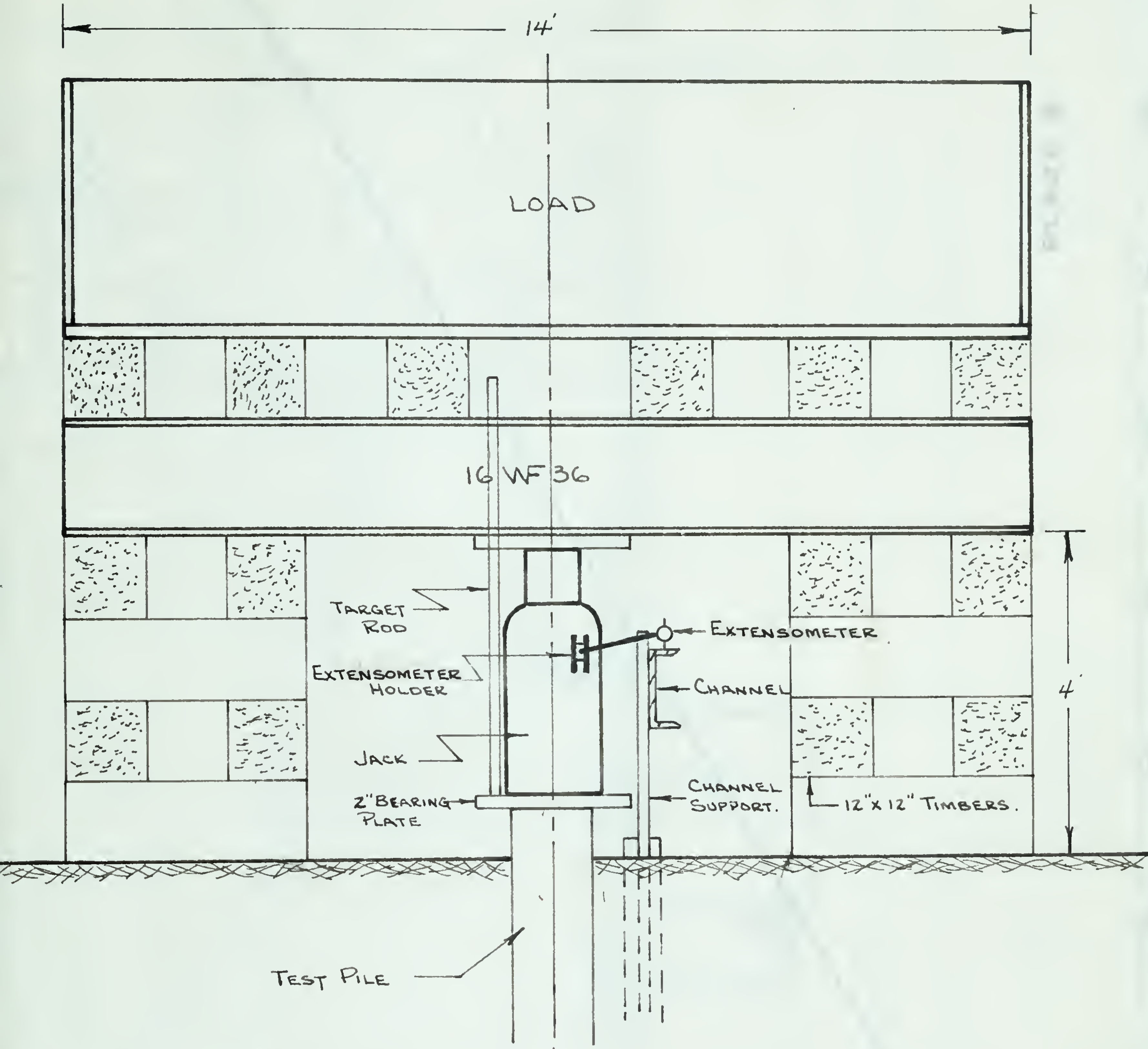


TYPICAL SECTIONS OF PILES TESTED

SCALES - HORIZONTAL 1"=2'
VERTICAL 1"=4'.

FEBRUARY 26, 1957.

PLATE 1.

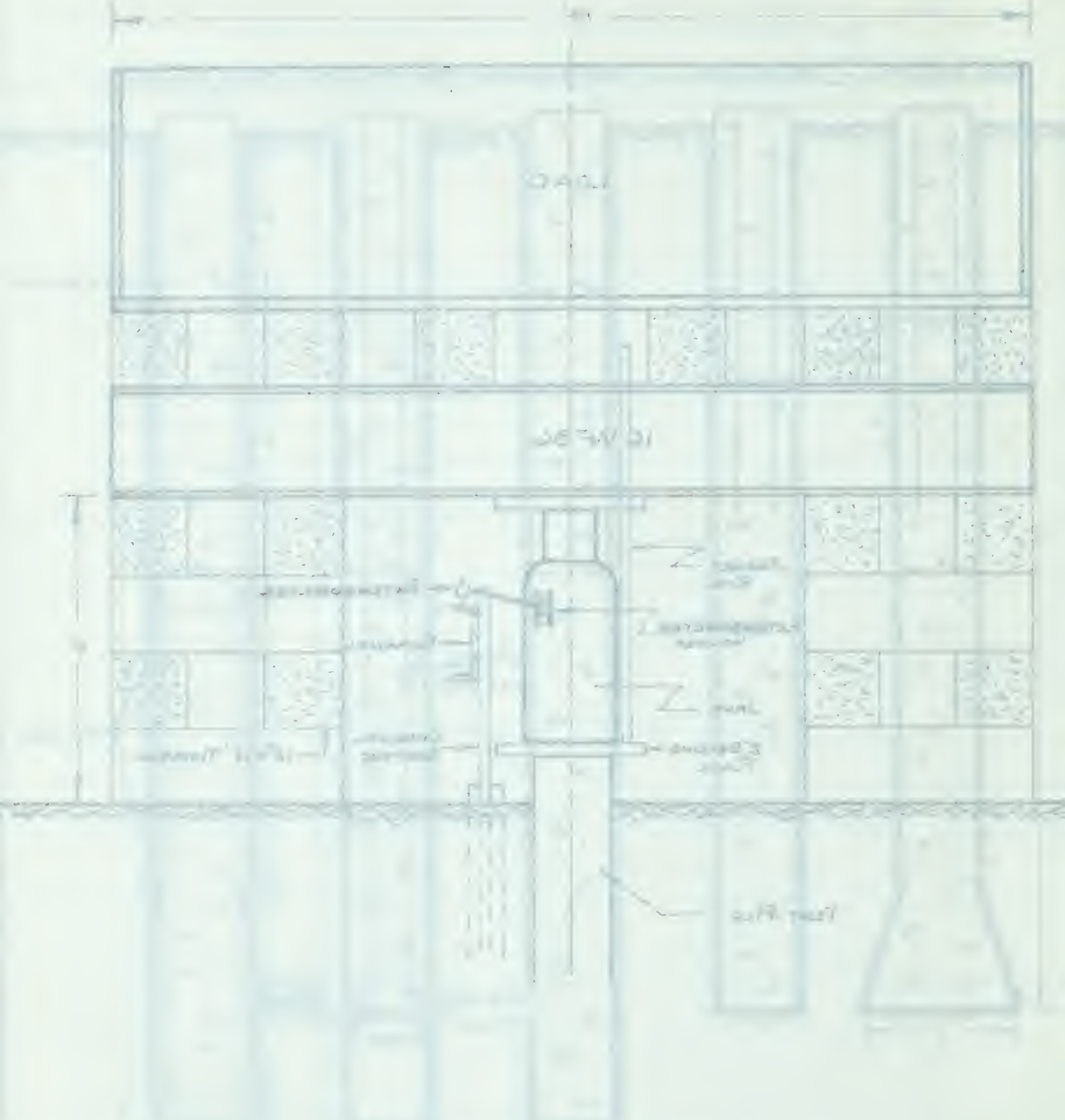


TEST PILE SETUP

NOT TO SCALE

FEB. 19, 1959.

PLATE 2.



TEST PILE SETUP

"Not to Scale"

PLATE 2

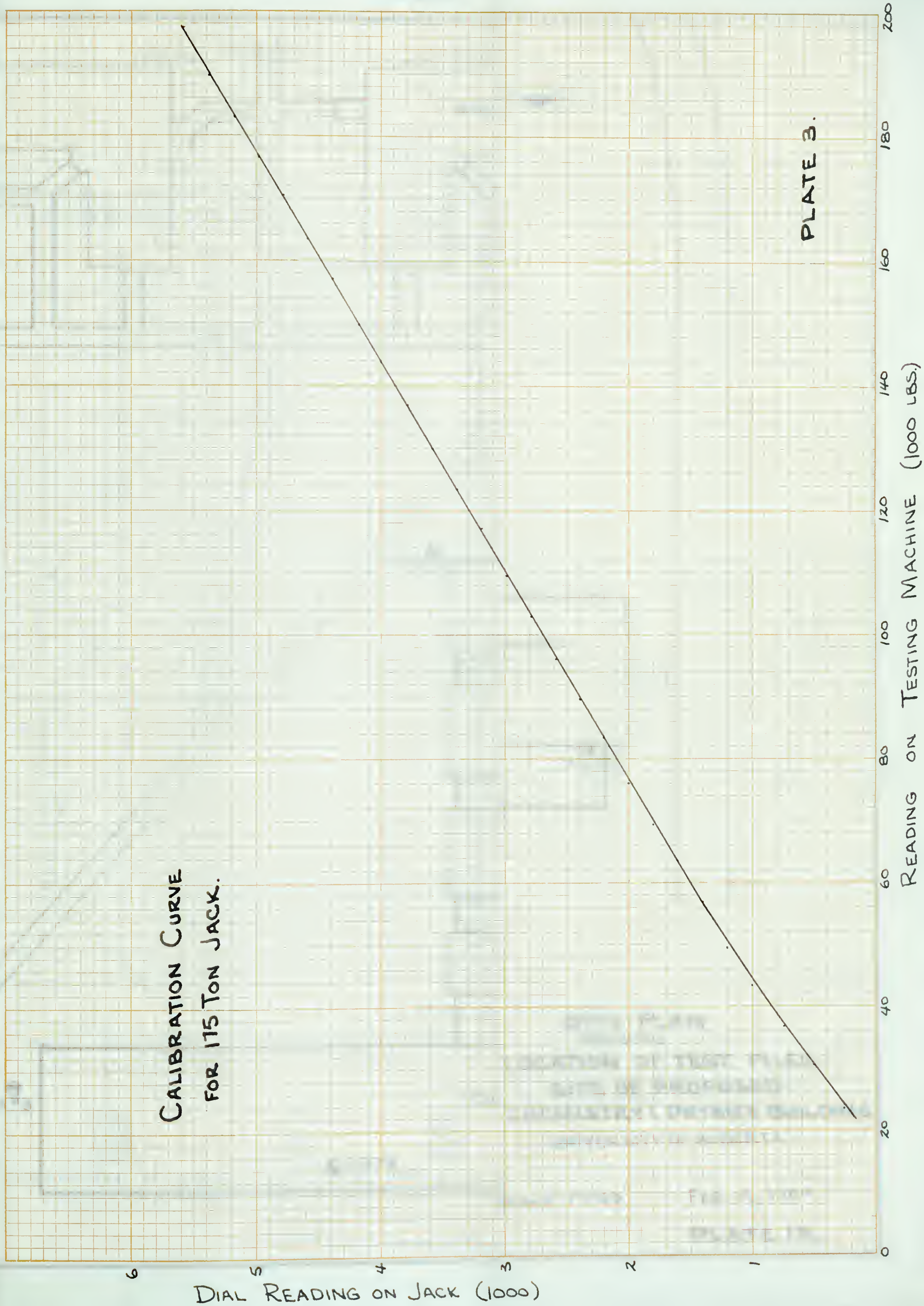
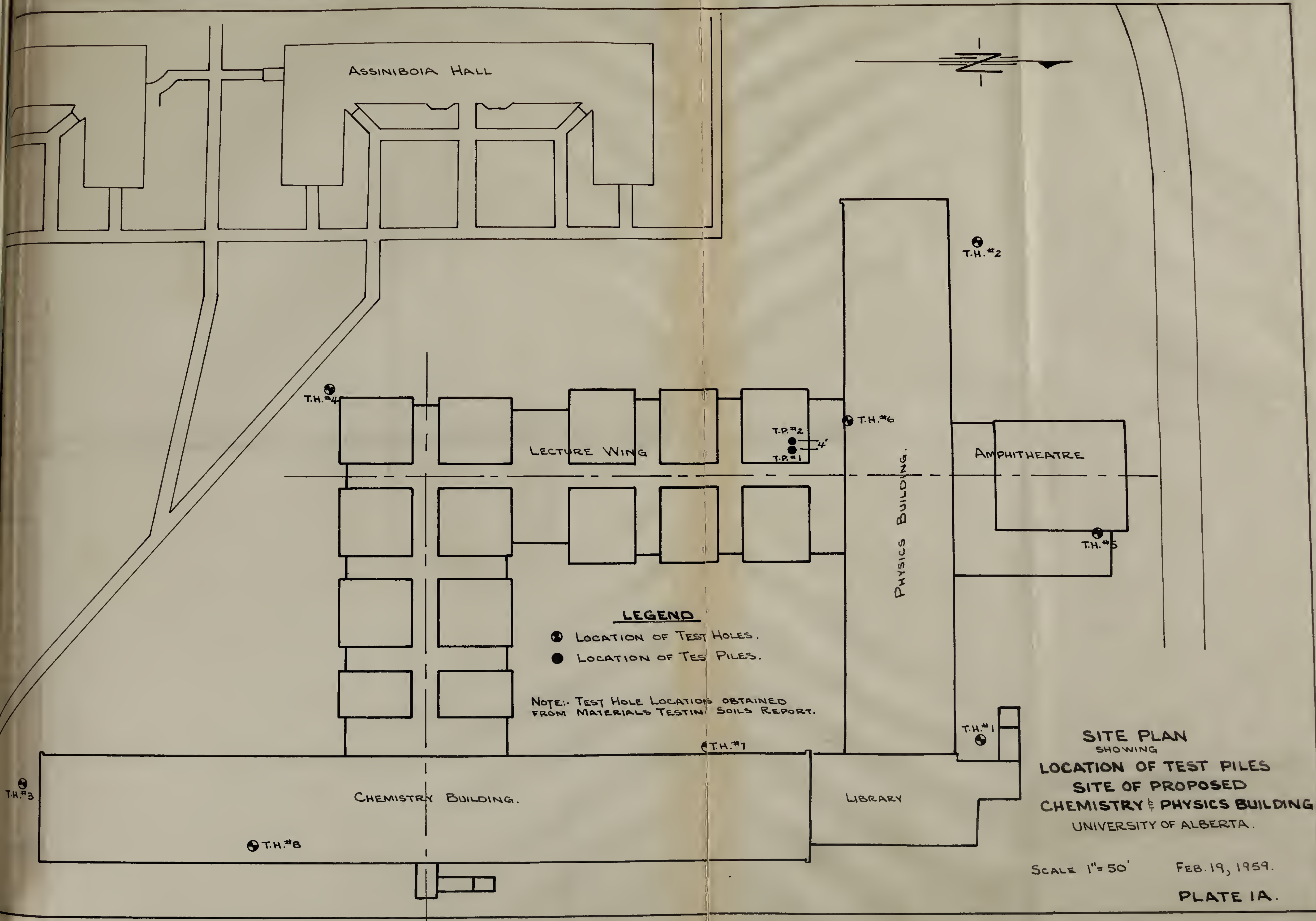


PLATE 3.
Calibration Curve
for 175 Ton Jack.
Reading on Testing Machine (1000 lbs.)
0
20
40
60
80
100
120
140
160
180
200
Dial Reading on Jack (1000)
1
2
3
4
5
6

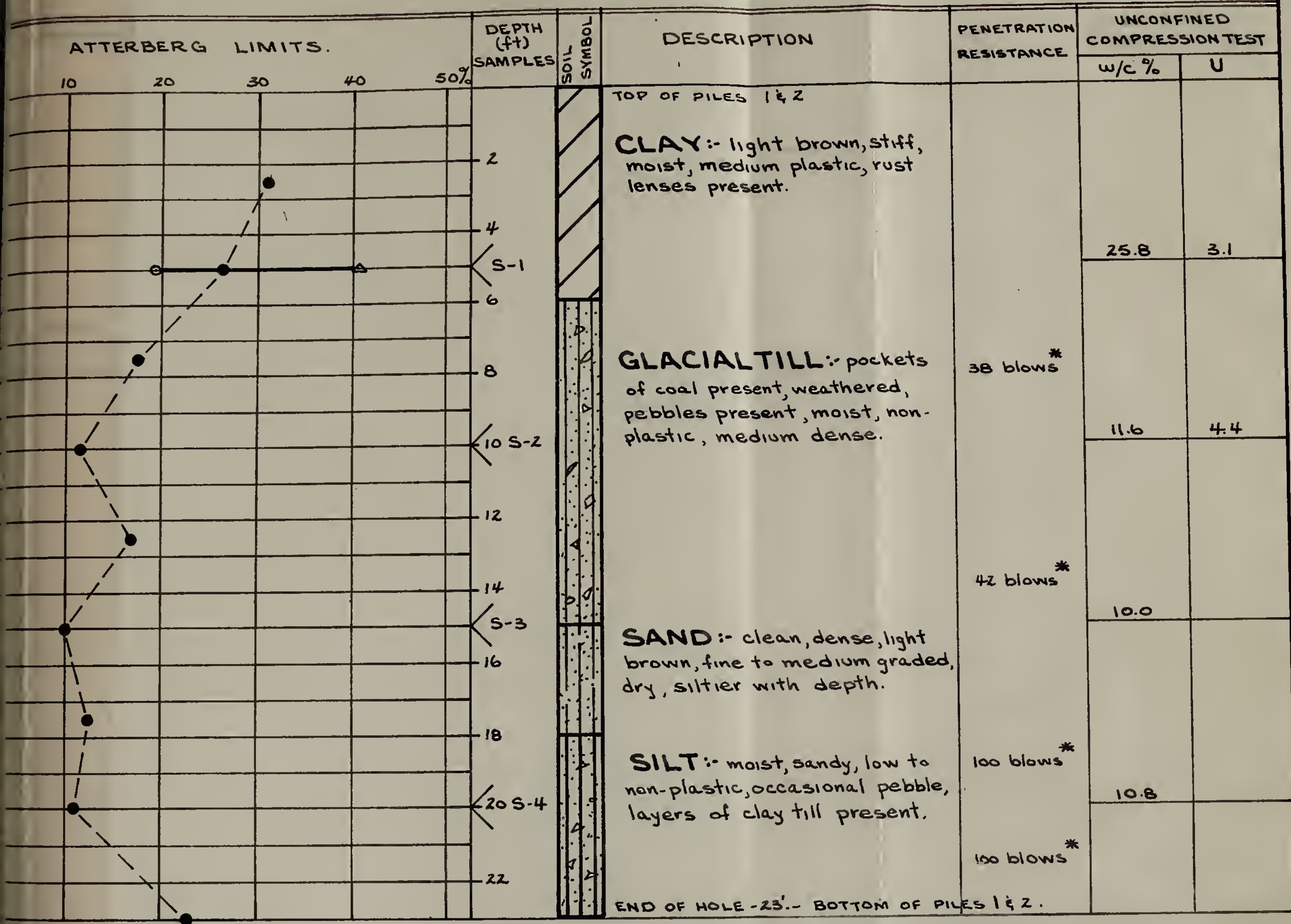

Assiniboia Hall
T.H. #2
T.H. #4
T.H. #6
T.P. #2
4'
T.P. #1
Lecture Wing
Amphitheatre
Physics Building.
T.H. #5
LEGEND
Location of Test Holes.
Location of Test Piles.
Note:- Test Hole Locations obtained from Materials Testing Soils Report.
T.H. #1
T.H. #7
T.H. #3
Chemistry Building.
Library
T.H. #8
Site Plan
showing
Location of Test Piles
Site of Proposed
Chemistry & Physics Building
University of Alberta.
Scale 1" = 50'
Feb. 19, 1959.
Plate IA.

SOIL PROFILE & SUMMARY of LABORATORY RESULTS.

PROJECT :- TEST PILE N° 2.

SITE :- PHYSICS & CHEMISTRY BUILDING,
UNIVERSITY of ALBERTA CAMPUS,
EDMONTON, ALTA.

PLATE 2A.



~ LEGEND ~

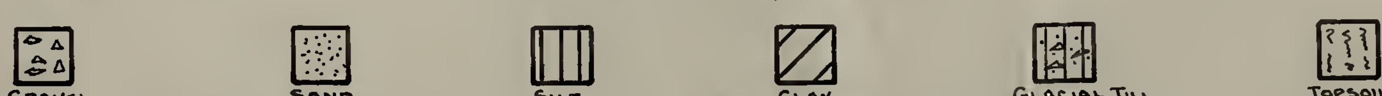
● & w/c - WATER CONTENT (PERCENT of DRY SOIL WEIGHT)

△ - LIQUID LIMIT.

⊙ - Plastic LIMIT.

U - UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ.FT)

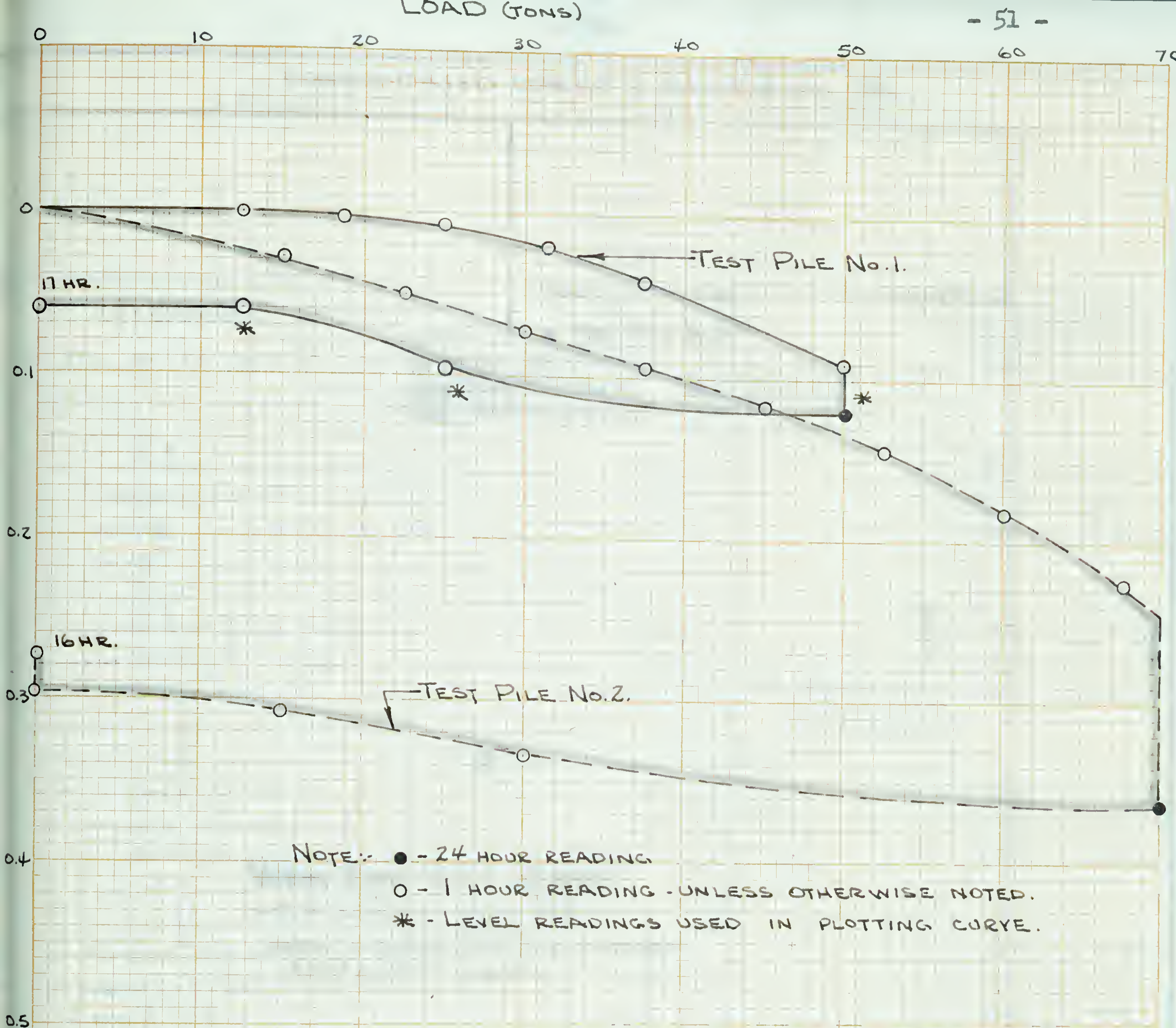
<S - SHELBY TUBE SAMPLES.



COMBINATIONS OF ABOVE SHOWN WITH PREDOMINATE SOIL TYPE IN HEAVY LINE & MODIFYING IN LIGHT LINE.

NOTE :- ABOVE LOG ALSO APPLIES TO TEST PILE N° 1.

* - STANDARD PENETRATION TEST RESULTS - TAKEN FROM TEST HOLE NO. 6 OF SOILS REPORT AS PREPARED BY MATERIALS TESTING LABORATORIES ON ABOVE SITE.



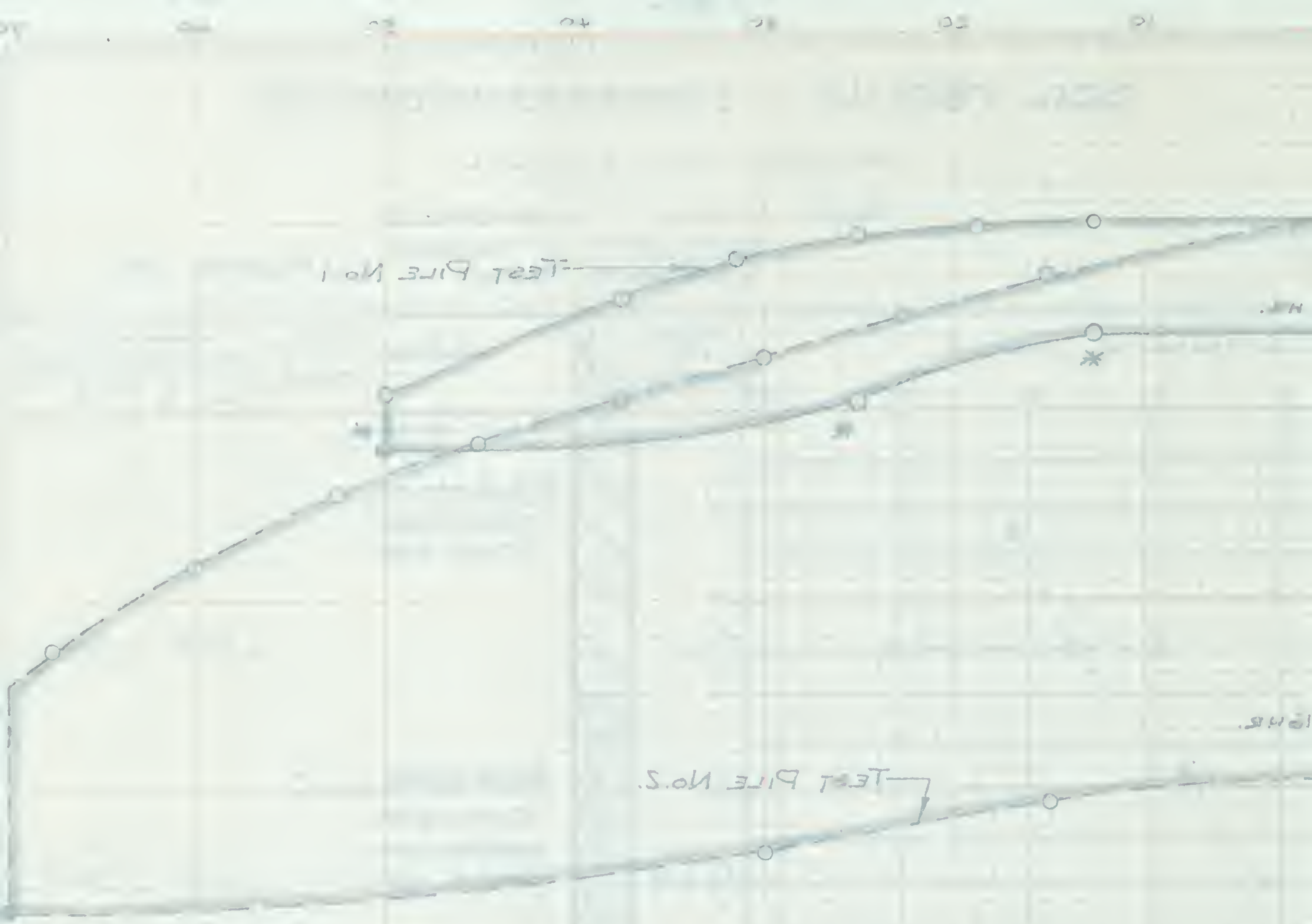
~ LOAD VS SETTLEMENT CURVES ~

— TEST PILE No 1.	- - TEST PILE No 2.
12" DIAMETER	12" DIAMETER
23' LONG.	23' LONG
2' DIAMETER BELL	NO BELL

SITE:- PROPOSED CHEMISTRY & PHYSICS BUILDING,
UNIVERSITY OF ALBERTA CAMPUS,
EDMONTON, ALTA.

FEBRUARY 2, 1959.

PLATE 3A.



Note:- ● - 24 hour reading
○ - 1 hour reading - unless otherwise noted
* - Level readings used in plotting curve.

LOAD vs SETTLEMENT CURVES

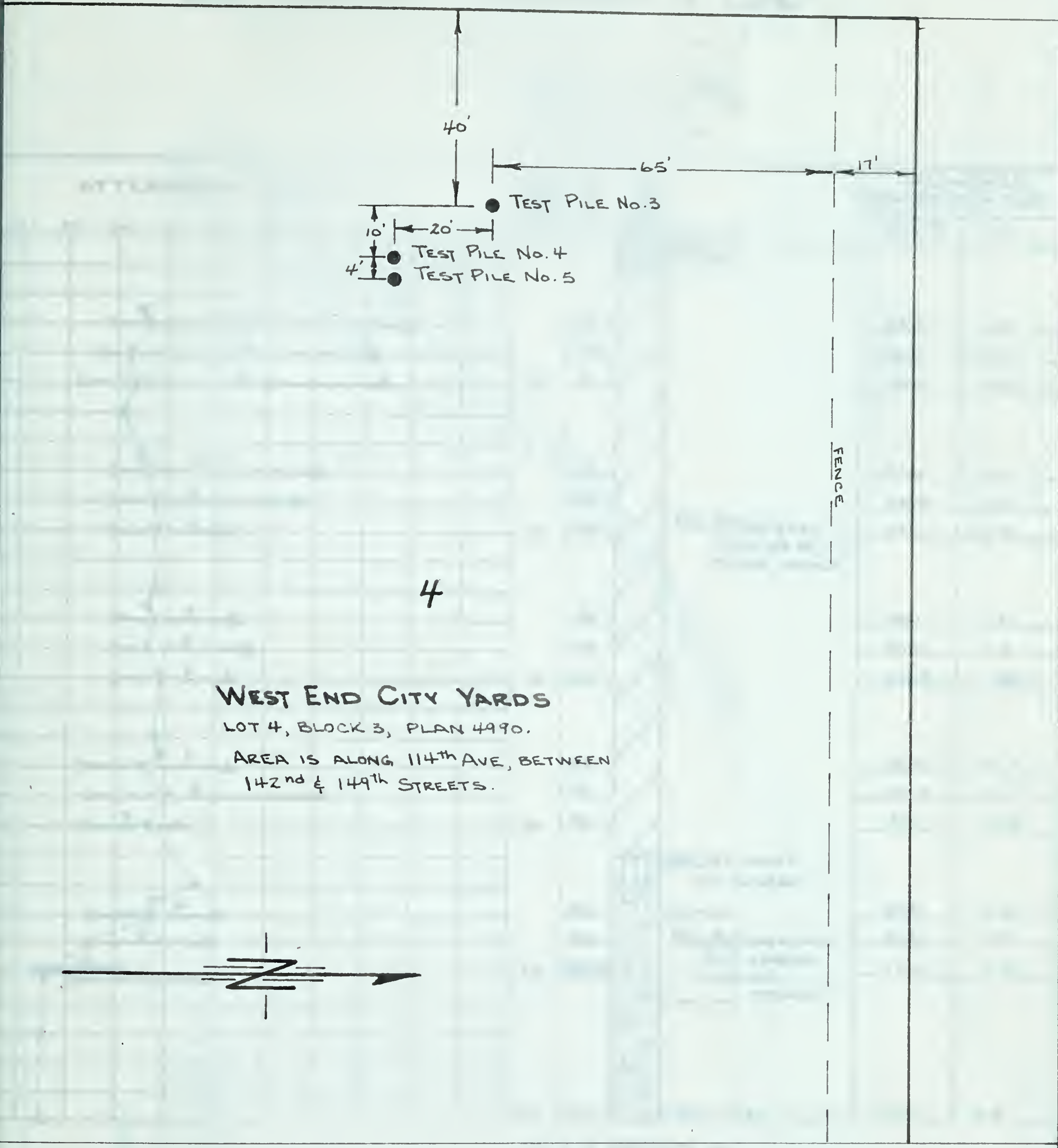
— Test Pile No 1. — — Test Pile No 2

Test Pile No 1	Test Pile No 2
12" Diameter	12" Diameter
23' Long.	23' Long
2' Diameter Bell	No Bell

Site:- Proposed Chemistry & Physics Building,
University of Alberta Campus,
Edmonton, Alta.

February 2, 1954. PLATE 3A.

CONSOLIDATED CONCRETE INDUSTRIES LTD.



SITE PLAN
SHOWING LOCATION OF TEST PILES No's 3, 4, & 5

SCALE - 1" = 30'

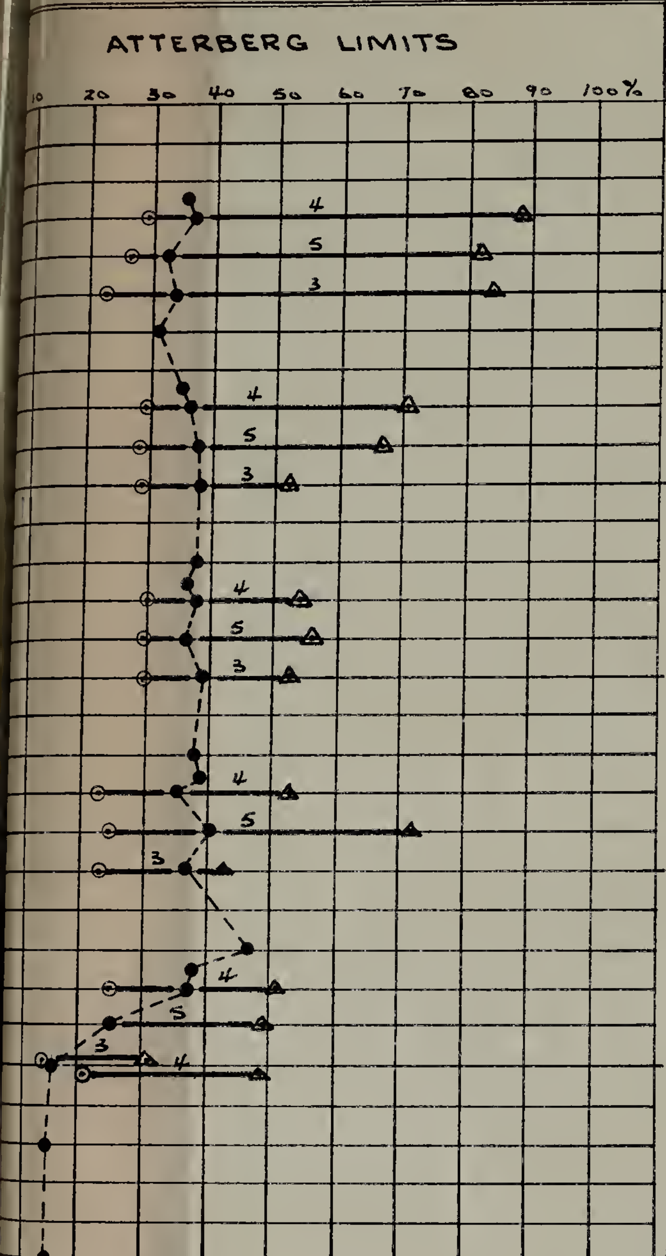
FEBRUARY 25th, 1959.

PLATE 1B.

SOIL PROFILE & SUMMARY of LABORATORY TESTS

PROJECT:- TEST PILES Nº 3, 4, & 5.

SITE:- WEST END CITY YARD,
114th AVE & 144th STREET,
EDMONTON, ALTA.



Depth (ft.)	Samples	Description	W/C %	U
		TOP OF PILES 3, 4 & 5.		
		FILL:- gravel, clay.		
	1_4		35.7	1.2
	1_5		32.6	1.1
5	1_3		32.6	2.0
	2_4		37.6	1.1
	2_5		38.3	1.0
10	2_3	CLAY:- highly plastic, mottled grey and brown, high salt content at 8', nugget structure to 12', firm, moist.	39.6	1.8
	3_4		38.1	1.2
	3_5		36.6	1.3
15	3_3		38.8	1.5
	4_4		36.4	0.9
	4_5		40.2	0.7
20	4_3		37.1	0.8
		SILT:- soft, grey, clayey, very moist, low to medium plastic, making water.		
	5_4	BOTTOM OF PILE 4.	27.8	1.2
	5_5	GLACIAL TILL:- soft at 22' becomes dense at 25', grey, medium plastic, coal, pea gravel and occasional cobble present.	26.6	1.4
25	5_3-6_4		17.6	2.4
		BOTTOM OF PILES 3 & 5.		
30	6_3	END OF HOLE - 30' - TEST PILE Nº 3.	14.0	2.5

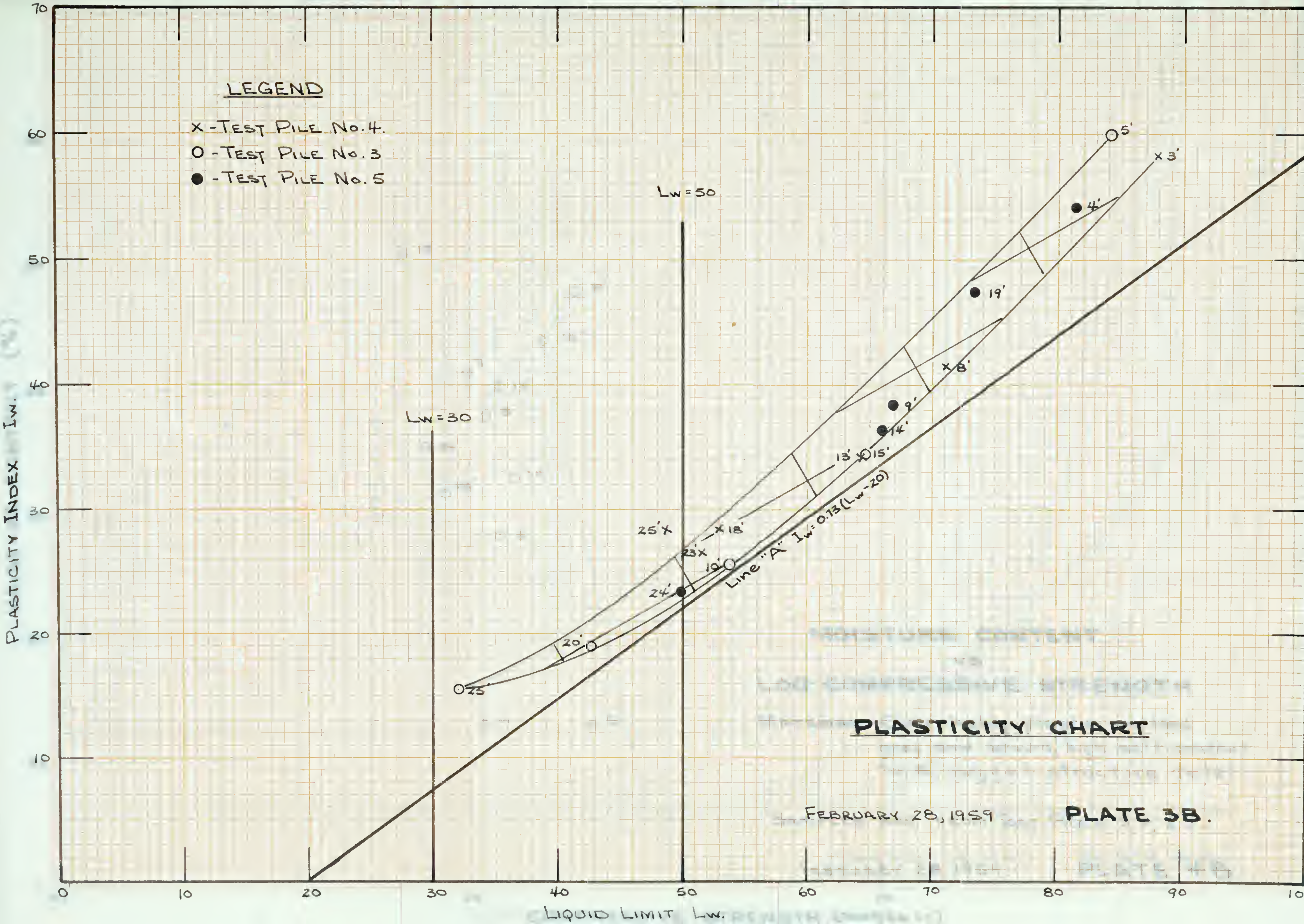
Column heads: UNCONFINED COMPRESSION TEST (W/C %, U); SOIL SYMBOL.

~ LEGEND ~

● & W/C - WATER CONTENT (PERCENT of DRY SOIL WEIGHT)
△ - LIQUID LIMIT.
U - UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ. FT.)
1_4 - INDICATES - SAMPLE Nº 1, TEST HOLE Nº 4.

COMBINATIONS OF ABOVE SHOWN WITH PREDOMINATE SOIL TYPE IN HEAVY LINE AND MODIFYING IN LIGHT LINE.

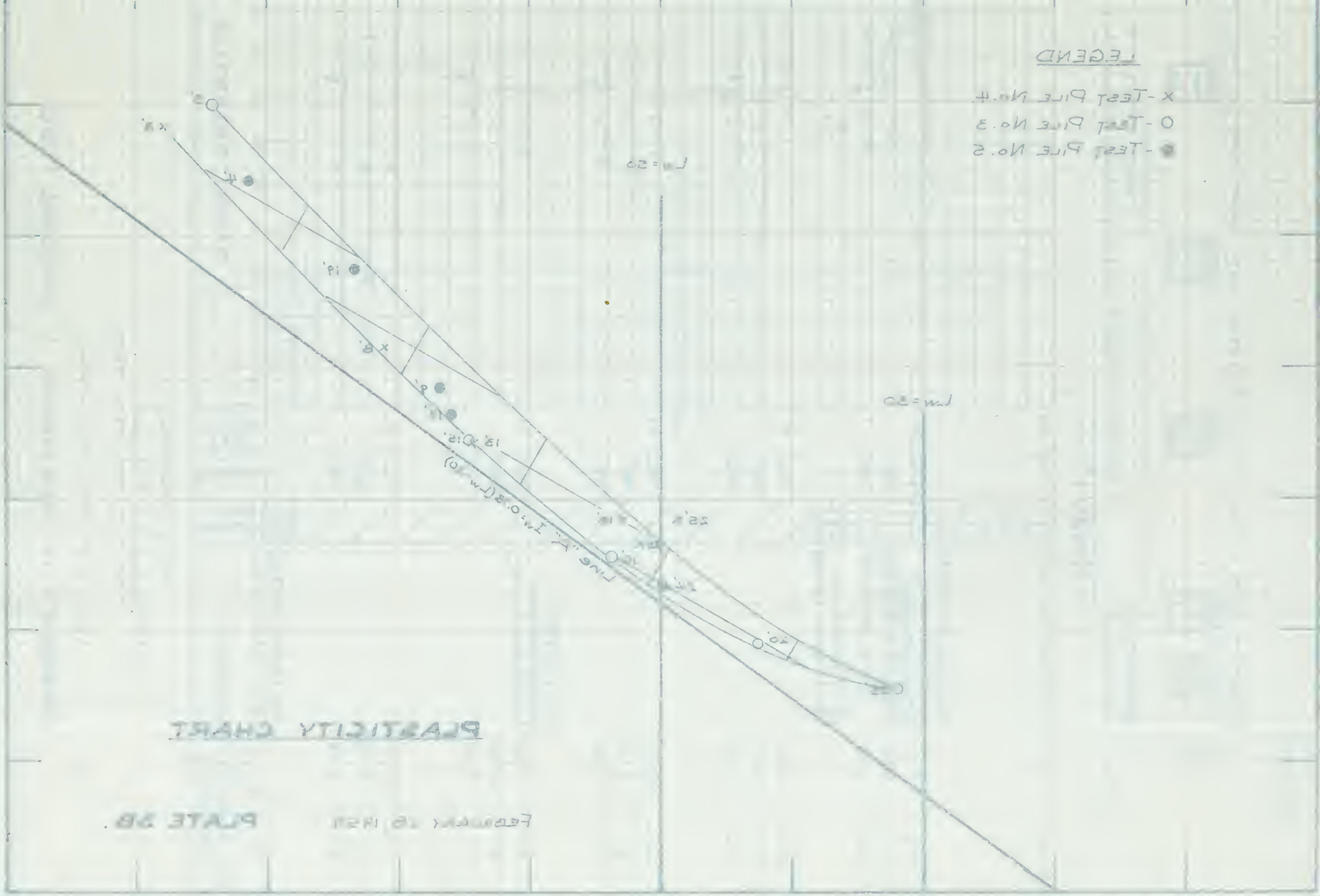
PLATE 2B.

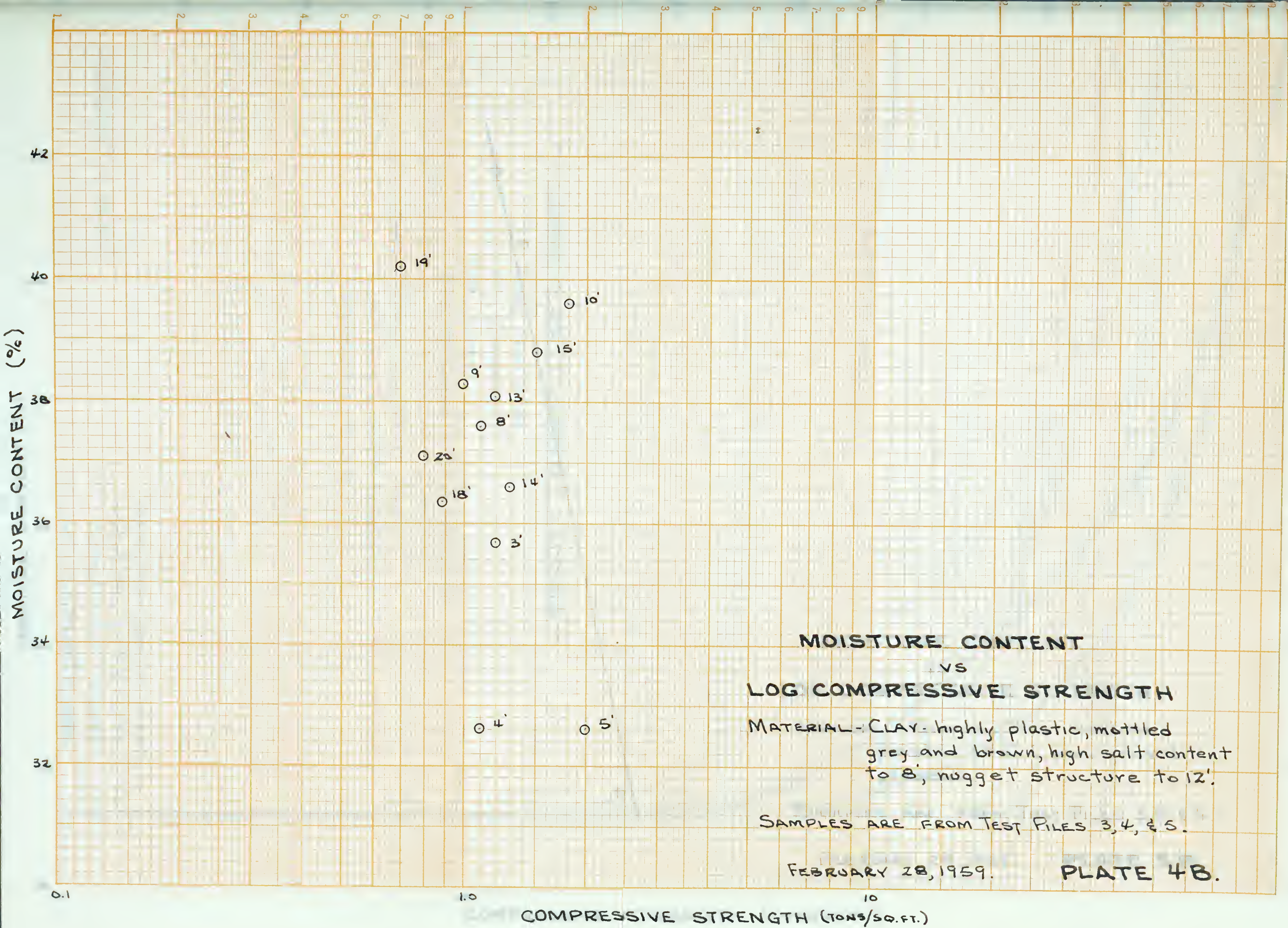


PLASTICITY CHART

FEBRUARY 28, 1959

PLATE 3B.


LEGEND
X - Test Pile No. 4
O - Test Pile No. 3
● - Test Pile No. 5
PLASTICITY CHART
PLATE 5B

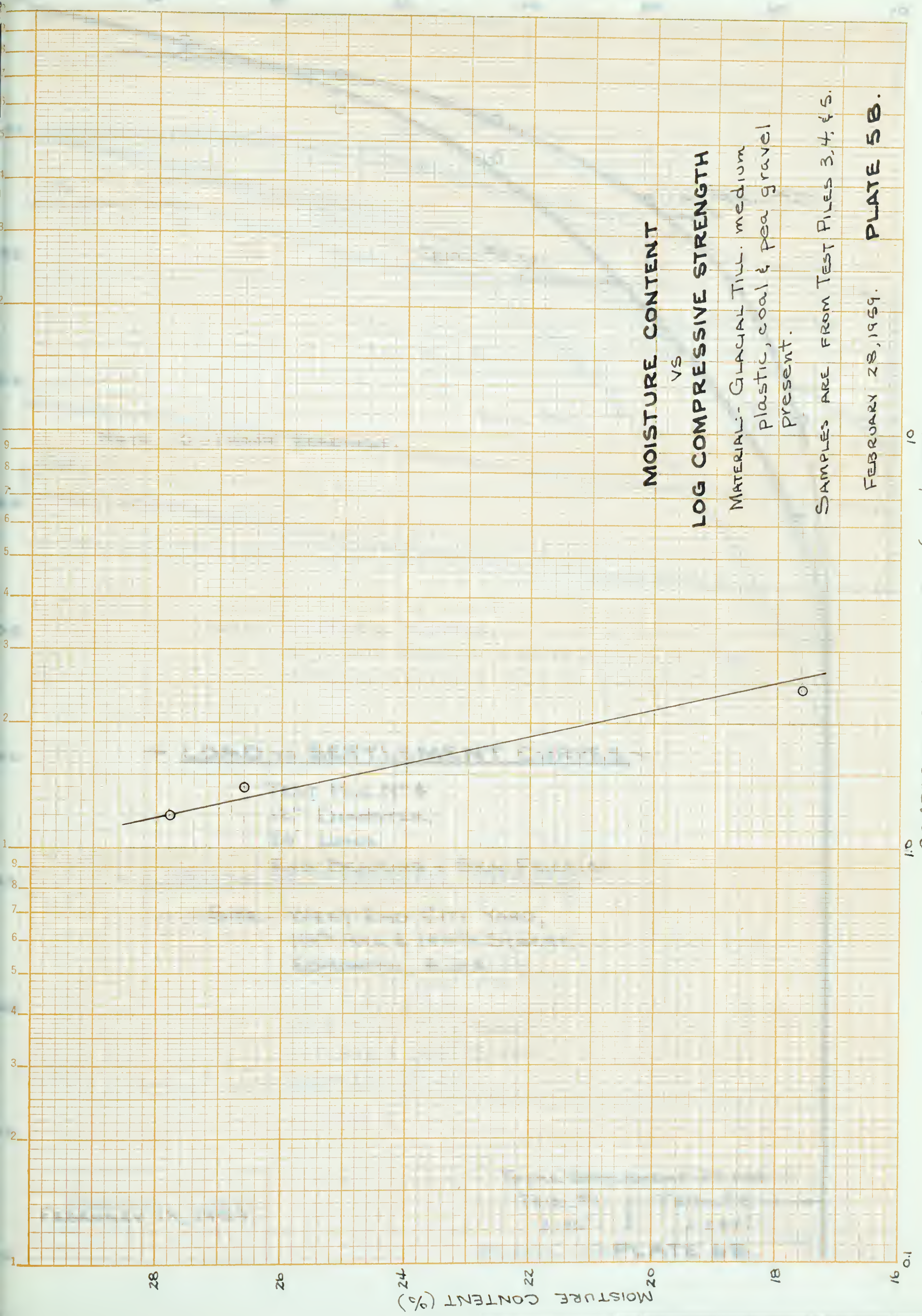

MOISTURE CONTENT
VS
LOG COMPRESSIVE STRENGTH
MATERIAL - CLAY - highly plastic, mottled grey and brown, high salt content to 8', nugget structure to 12'.
SAMPLES ARE FROM TEST PILES 3, 4, & 5.
FEBRUARY 28, 1959.
PLATE 4B.
MOISTURE CONTENT (%)
42
40
38
36
34
32
COMPRESSIVE STRENGTH (TONS/SQ.FT.)
0.1
1.0
10
19'
10'
15'
9'
13'
8'
20'
14'
18'
3'
4'
5'

MOISTURE CONTENT
vs
LOG COMPRESSIVE STRENGTH

Material - Clay - highly plastic, mottled grey and brown, high silt content to 8, nugget structure to 12.

Samples are from Test Pits 3, 4, & 5.

February 28, 1959. **PLATE 48.**


MOISTURE CONTENT
VS
LOG COMPRESSIVE STRENGTH
MATERIAL:- GLACIAL TILL. medium plastic, coal & pea gravel present.
SAMPLES ARE FROM TEST PILES 3, 4 & 5.
FEBRUARY 28, 1959.
PLATE 5B.
0.1
1.0
10
COMPRESSIVE STRENGTH (TONS/SQ.FT.)
28
26
24
22
20
18
16
MOISTURE CONTENT (%)

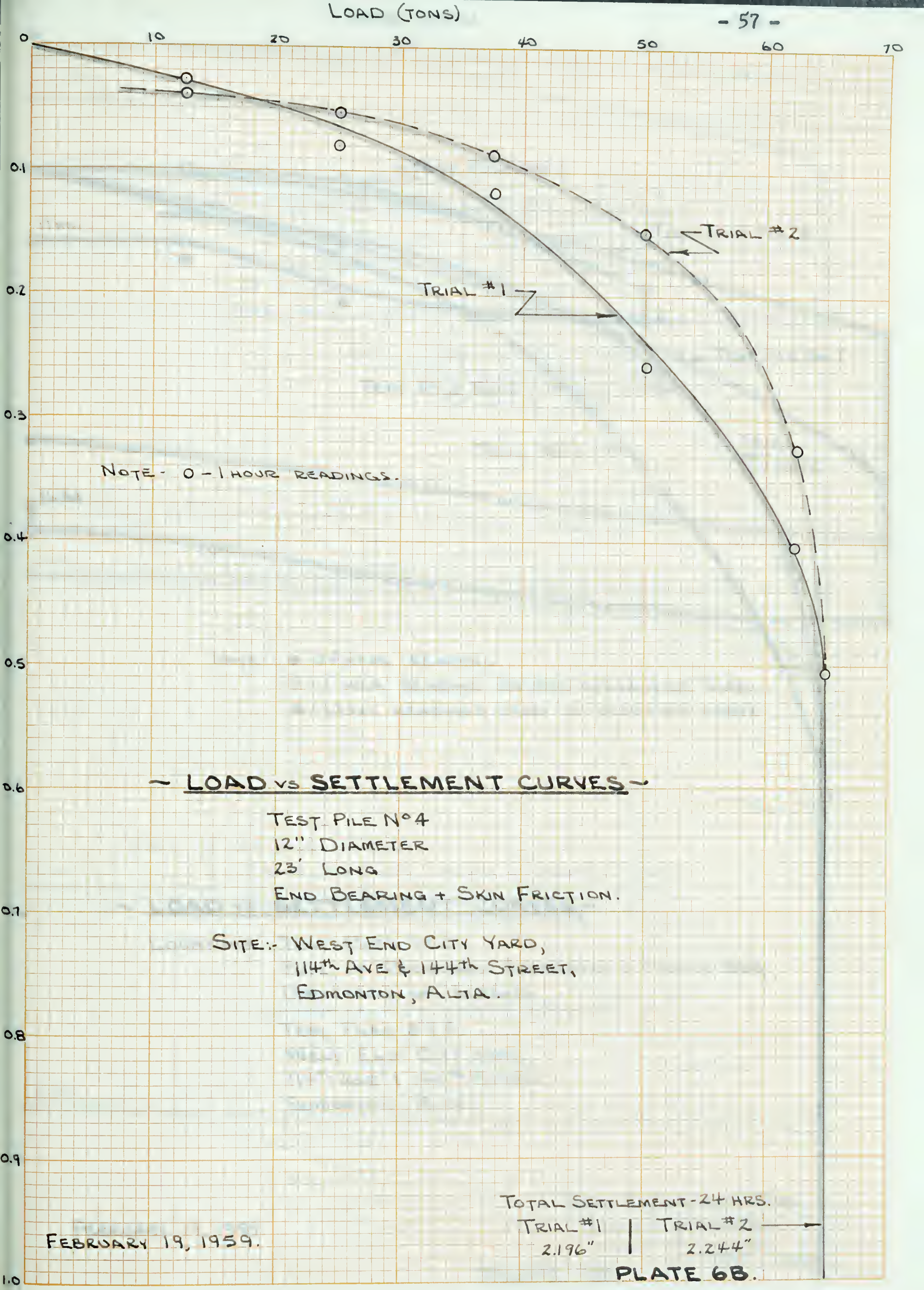
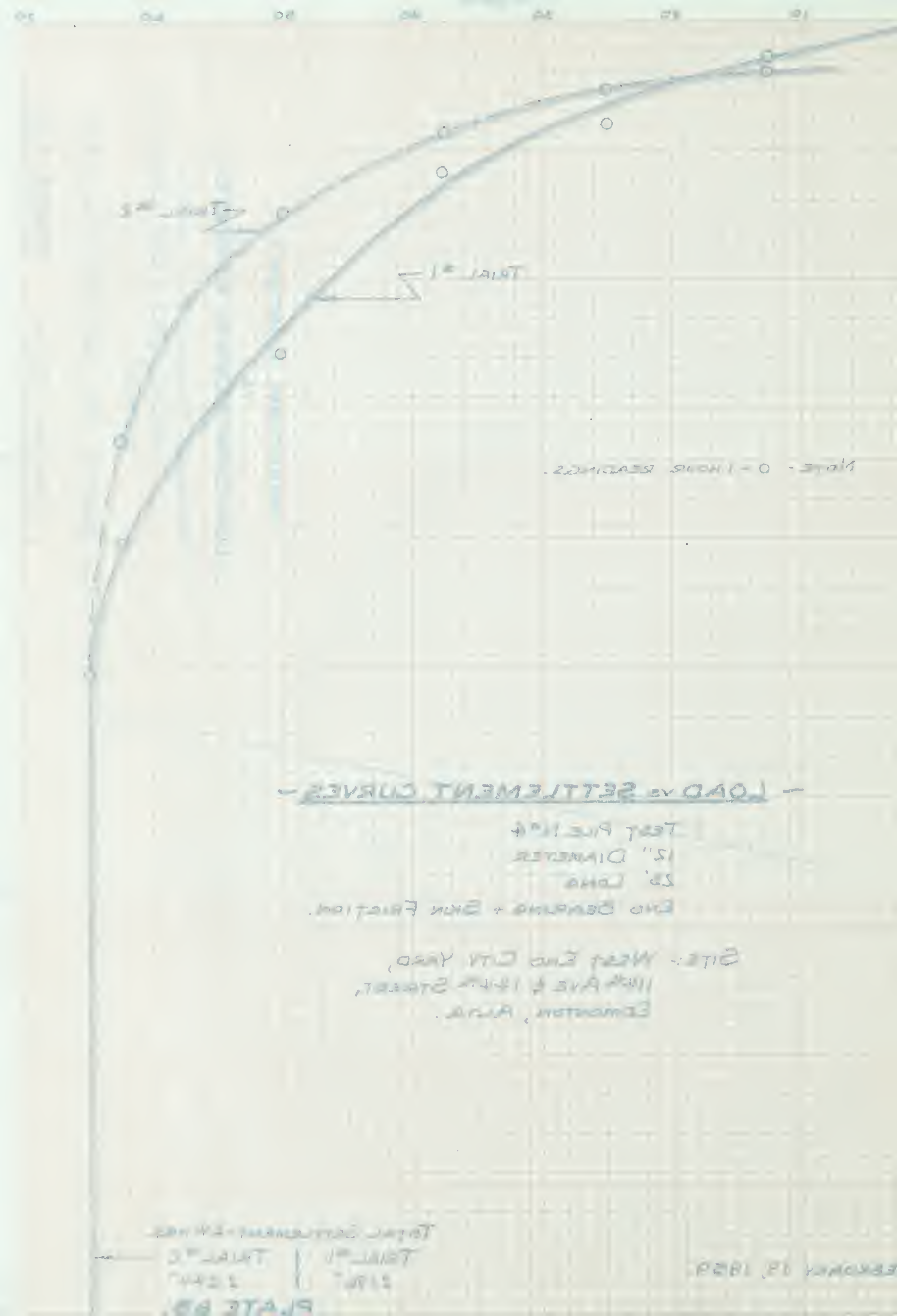
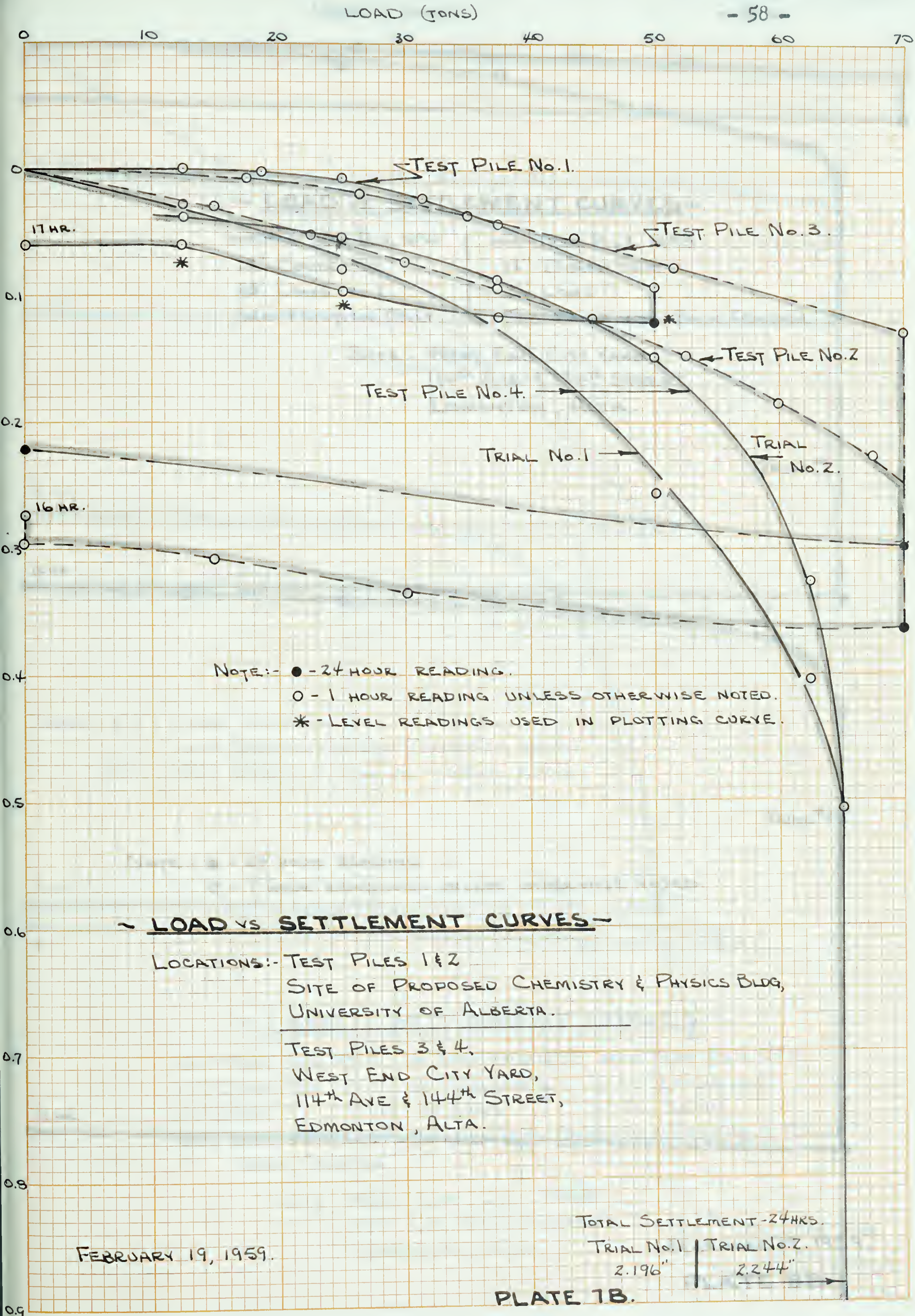

LOAD (TONS)
0
10
20
30
40
50
60
70
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
TRIAL #2
TRIAL #1
NOTE - 0 - 1 HOUR READINGS.
~ LOAD vs SETTLEMENT CURVES ~
TEST PILE N°4
12" DIAMETER
23' LONG
END BEARING + SKIN FRICTION.
SITE:- WEST END CITY YARD,
114th AVE & 144th STREET,
EDMONTON, ALTA.
TOTAL SETTLEMENT - 24 HRS.
TRIAL #1 | TRIAL #2
2.196" | 2.244"
FEBRUARY 19, 1959.

PLATE 6B.


TRIAL #2
TRIAL #1
NOTE - O = 1 HOUR READINGS.
— LOAD vs SETTLEMENT CURVES —
TEST PILE No 4
12" DIAMETER
25' LONG
END BEARING + SKIN FRICTION.
SITE:- WEST END CITY YARD,
116th AVE & 144th STREET,
EDMONTON, ALTA.
TRIAL #1
TRIAL #2
FEBRUARY 13, 1959.
PLATE 63.



~ LOAD VS SETTLEMENT CURVES ~

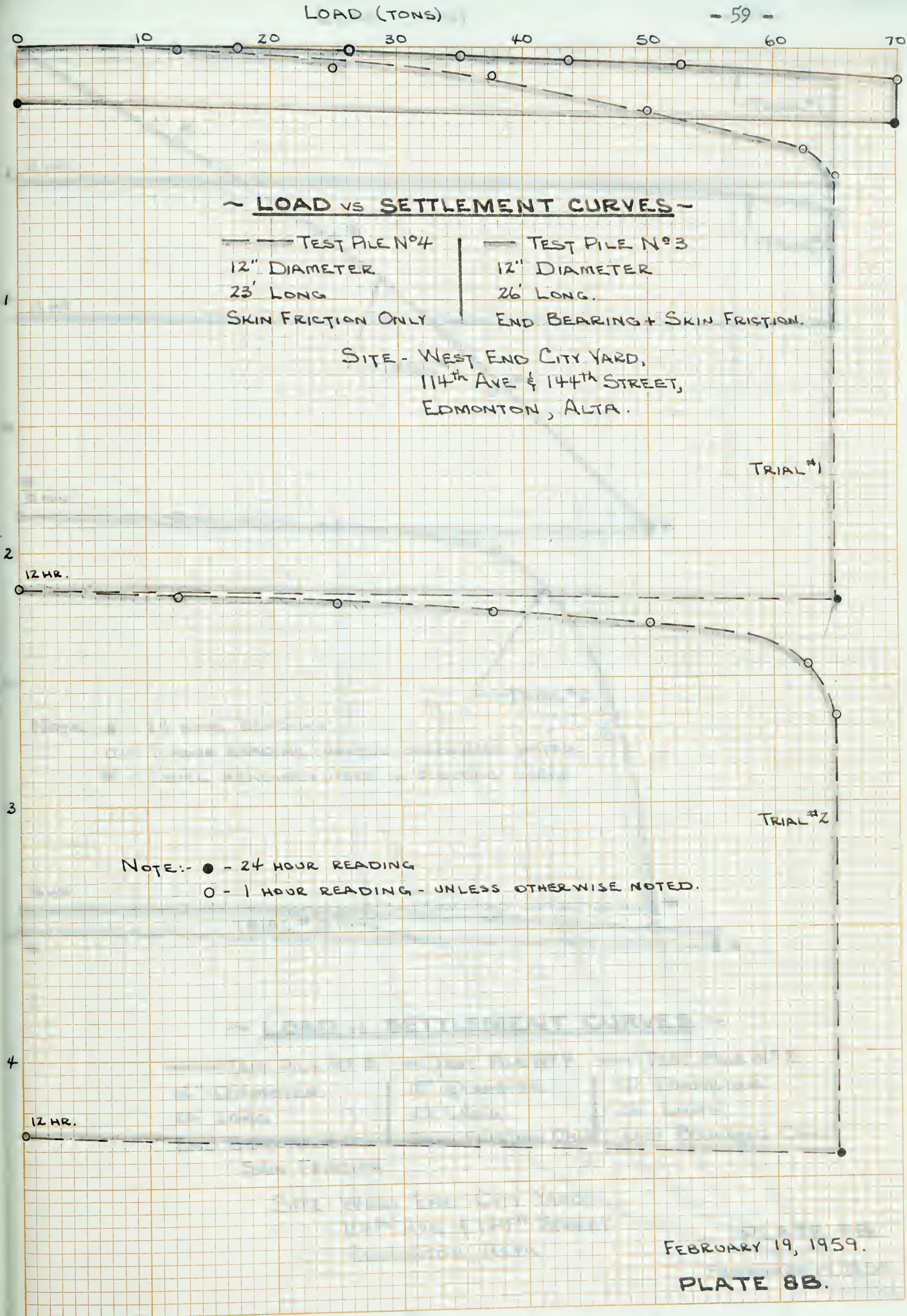
LOCATIONS:- TEST PILES 1 & 2
SITE OF PROPOSED CHEMISTRY & PHYSICS BLDG,
UNIVERSITY OF ALBERTA.

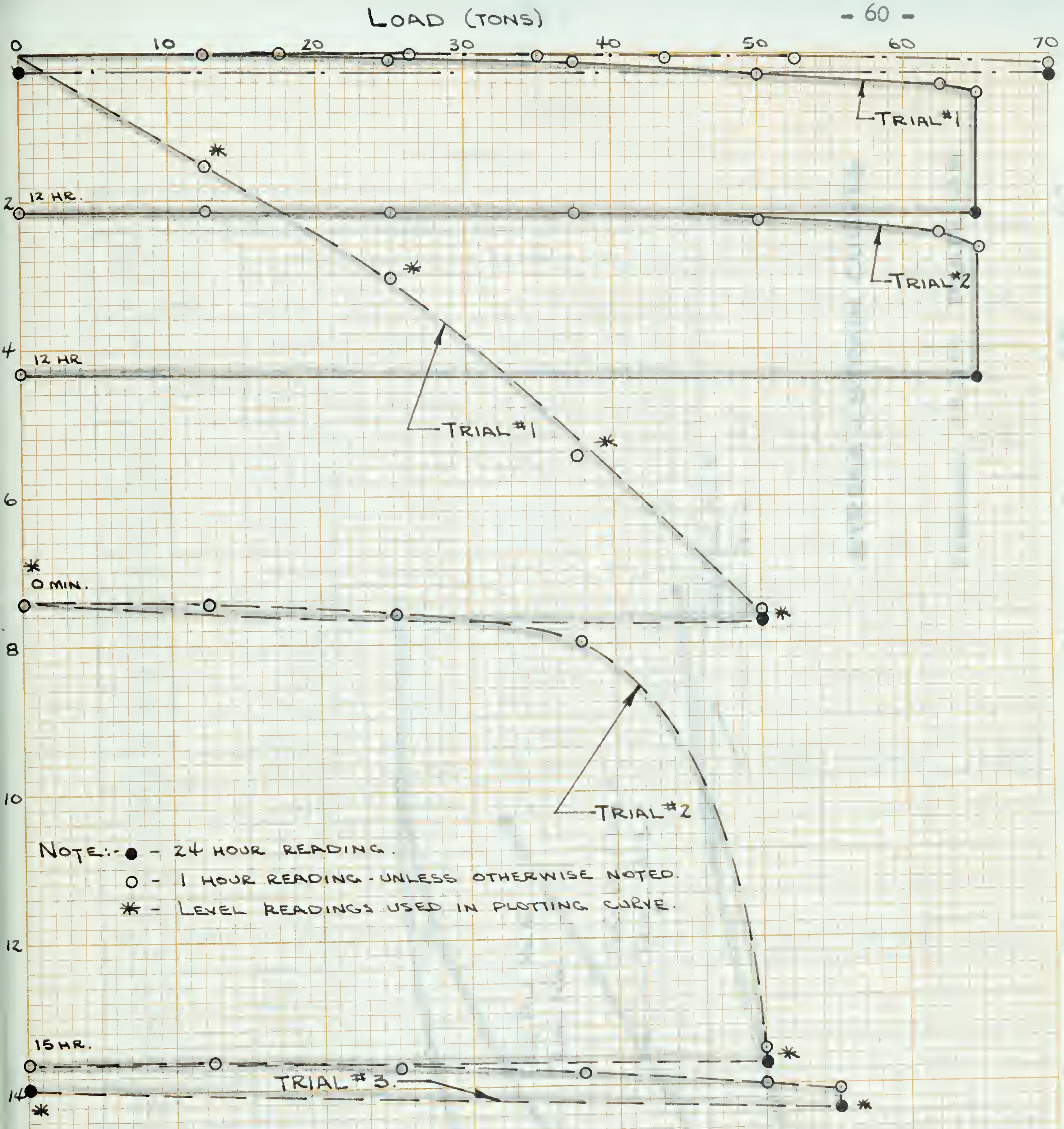
TEST PILES 3 & 4.
WEST END CITY YARD,
114th AVE & 144th STREET,
EDMONTON, ALTA.

FEBRUARY 19, 1959.

PLATE 7B.


Test Pile No. 1
Test Pile No. 3
Test Pile No. 2
Test Pile No. 4
Trial No. 1
Trial No. 2
Note:- ● - 24 hour reading.
O - 1 hour reading unless otherwise noted.
* - Level readings used in plotting curve.
LOAD vs SETTLEMENT CURVES
Locations: Test Piles 1 & 2,
Site of Proposed Chemistry & Physics Bldg,
University of Alberta.
Test Piles 3 & 4,
West End City Yards,
114th Ave & 114th Street,
Edmonton, Alta.
February 17, 1959.
PLATE 18


LOAD (TONS)
0
10
20
30
40
50
60
70
1
2
3
4
~ LOAD vs SETTLEMENT CURVES ~
TEST PILE No 4
12" DIAMETER
23' LONG
SKIN FRICTION ONLY
TEST PILE No 3
12" DIAMETER
26' LONG.
END BEARING + SKIN FRICTION.
SITE - WEST END CITY YARD,
114th AVE & 144th STREET,
EDMONTON, ALTA.
TRIAL #1
12 HR.
TRIAL #2
NOTE:- ● - 24 HOUR READING
O - 1 HOUR READING - UNLESS OTHERWISE NOTED.
12 HR.
FEBRUARY 19, 1959.
PLATE 8B.



~ LOAD vs SETTLEMENT CURVES ~

—·— TEST PILE Nº 3.	—— TEST PILE Nº 4	— — TEST PILE Nº 5
12" DIAMETER.	12" DIAMETER	12" DIAMETER
26' LONG.	23' LONG	26' LONG.
END BEARING + SKIN FRICTION.	SKIN FRICTION ONLY	END BEARING ONLY.

SITE - WEST END CITY YARD
114th AVE & 144th STREET.
EDMONTON, ALTA.

PLATE 9B.

FEBRUARY 19, 1959.

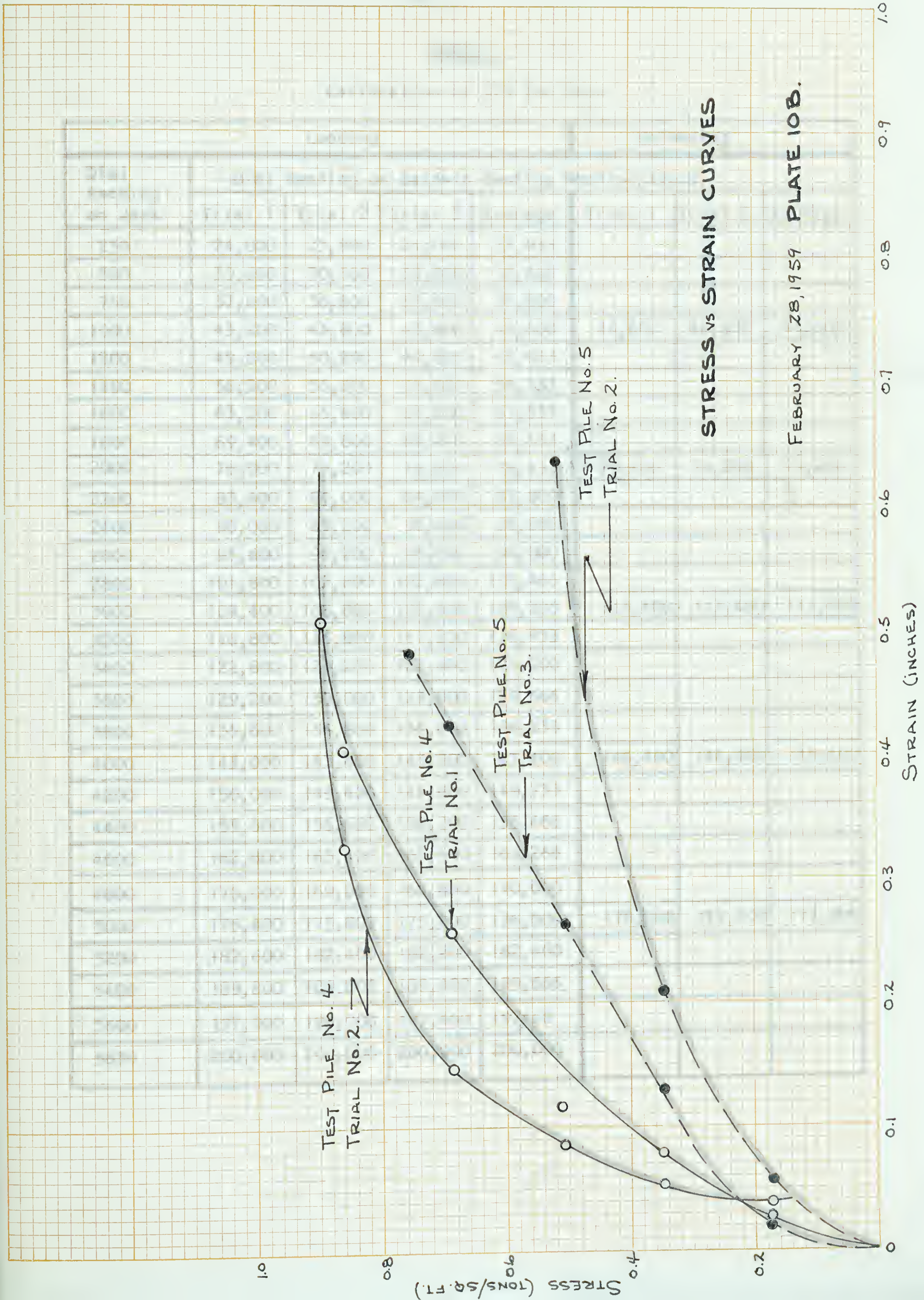

STRESS vs STRAIN CURVES
PLATE 10B.
FEBRUARY 28, 1959
TEST PILE No. 4
TRIAL No. 2.
TEST PILE No. 4
TRIAL No. 1
TEST PILE No. 5
TRIAL No. 3.
TEST PILE No. 5
TRIAL No. 2.
STRAIN (INCHES)
STRESS (TONS/SQ.FT.)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
0.2
0.4
0.6
0.8
1.0

TABLE I

Calibration of 175 Ton Jack

Loading					Unloading		
Dial Reading on Jack	Dial Reading on Baldwin Testing Machine (lbs)						
	Trial 1	Trial 2	Trial 3	Average	Trial 1	Trial 2	Average
250	24,000	23,800	24,000	23,933			
500	30,600	30,600	30,800	30,666			
750	37,000	36,800	36,600	36,800			
1000	43,200	43,800	43,800	43,600	44,600	44,600	44,600
1200	49,800	50,200	49,800	49,933			
1400	56,200	56,600	56,800	56,333			
1600	63,200	63,600	63,800	63,533			
1800	69,400	69,800	69,400	69,533			
2000	76,000	76,200	76,200	76,133	79,200	79,200	79,200
2200	83,400	84,000	84,000	83,800			
2400	90,000	89,200	90,400	89,866			
2600	95,600	96,200	96,200	96,000			
2800	101,800	102,800	102,800	102,466			
3000	109,400	108,800	109,400	109,200	113,400	113,400	113,400
3200	116,800	116,800	117,200	116,933			
3400	122,800	123,200	123,600	123,200			
3600	129,200	130,000	129,800	129,666			
3800	135,600	136,800	136,800	136,333			
4000	143,000	143,400	143,800	143,400	146,400	146,400	146,400
4200	150,000	149,400	149,800	149,733			
4400	155,600	156,600	156,200	156,466			
4600	162,800	163,400	163,600	163,266			
4800	170,000	169,800	169,800	170,000			
5000	176,400	175,800	175,800	176,000	178,200	177,600	177,900
5200	182,600	182,400	182,200	182,400			
5400	189,800	189,800	189,400	189,666			
5600	197,000	198,000	197,400	197,466			
5850	200,000	200,000	200,000	200,000			

TABLE 2

RESULTS OF LOAD TEST ON PILE #1

Location: Site of Physics-Chemistry Building,
University of Alberta Campus
Edmonton, Alberta

Pile: 12" Ø - 2' Ø bell - 23' deep

Design Load = 25 tons

% of Design Load	Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	Level Readings (ft.) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
25	6.25	nil	0.300						2.572				2.572	0
50	12.5	300	0.300	0.300	0.300	0.300	0.300	0.00	2.572				2.572	0
75	18.75	750	0.3018	0.3018	0.3018	0.3018	0.3018	0.0018	2.572				2.572	0
100	25	1,190	0.3052	0.3072	0.3075	0.3080	0.3085	0.0085	2.572				2.572	0
125	31.25	1,560	0.3135	0.3171	0.3188	0.3202	0.3208	0.0208	2.573				2.573	0.012
150	37.5	1,940	0.3278	0.3341	0.3372	0.3391	0.3410	0.0410	2.573				2.575	0.036
200	50	2,700	0.3590 4 1/2 hr 0.4182	14 hr 0.4469	16 hr 0.4627	19 hr 0.4748	0.3902 24 hr 0.4840	0.0902 0.1840	2.575				2.577 24 hr 2.582	0.060 0.120*
Rebound: 50	25	1,190	0.4705					0.1705	2.580					0.096*
25	12.5	300	0.4630					0.1630	2.577					0.060*
0	0	0	0.4320	0.3770		3 hr 0.3641	17 hr 0.3578	0.0578	2.577				17 hr 2.577	0.060

Note: *Indicates level readings used in plotting load-settlement curve.

TABLE 3

RESULTS OF LOAD TEST ON PILE #2

Location: Site of Physics-Chemistry Buildihg, University of Alberta Campus, Edmonton, Alberta

Pile: 12" Ø - 2' Ø Bell - 23' deep

Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	Level Readings (ft) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
7.50	nil	0.5906						3.260					
15	500	0.6130	0.6170	0.6182	0.6191	0.6196	0.0290	3.260				3.260	0
22.5	1,050	0.6324	0.6381	0.6391	0.6410	0.6412	0.0506	3.262				3.262	0.024
30	1,500	0.6512	0.6562	0.6591	0.6608	0.6618	0.0712	3.264				3.264	0.048
37.5	1,950	0.6709	0.6750	0.6781	0.6815	0.6830	0.0924	3.265				3.266	0.072
45	2,400	0.6930	0.6989	0.7041	0.7069	0.7084	0.1178	3.268				3.268	0.096
52.5	2,850	0.7190		0.7341	0.7350	0.7351	0.1445	3.269		3.270		3.270	0.120
60	3,300	0.7500	0.7607	0.7660	0.7714	0.7735	0.1829	3.271				3.271	0.132
67.5	3,750	0.7907	0.8004	0.8110	0.8150	0.8182	0.2276	3.274				3.274	0.168
70	3,900	0.8389		0.8548	5 hr 0.8900	16 1/2 hr 0.9220	0.3617			3.276	3.277	5 hr 3.284	0.348
					18 1/2 hr 0.9381	24 hr 0.9523					16 1/2 hr 3.288	24 hr 3.289	
Rebound: 30	1,500	0.9245					0.3339	3.286					0.312
15	500	0.8990					0.3084	3.284					0.288
0	0	0.8860			5 1/2 hr 0.8712	16 hr 0.8635	0.2729	3.283		5 1/2 hr 3.283		16 hr 3.281	0.252

TABLE 4

RESULTS OF LOAD TEST ON PILE #4

Location: West End City Yard
114th Ave. & 144th St.
Edmonton, Alberta

Trial #1
Pile: 12" Ø - 23' deep - no bell
No end bearing

Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n. (ins)	Level Readings (ft) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n
0	0	0.1463						3.270					
12.5	300	0.1620	0.1690	0.1715	0.1732	0.1733	0.0281	3.270			3.271	3.272	0.024
25.0	1,190	0.2025	0.2150	0.2214	0.2237	0.2262	0.0799	3.274	3.275			3.276	0.072
37.5	1,940	0.2340	0.2426	0.2520	0.2578	0.0630	0.1167	3.277	3.278	3.280		3.281	0.132
50	2,700	0.2985	0.3487	0.3918	0.4047	0.4033	0.2570	3.284	3.286	3.288	3.290	3.290	0.240
62.5	3,450	0.4615	0.4873	0.5115	0.5320	0.5497	0.4034	3.291	3.292	3.293	3.296	3.300	0.360
65	3,600	0.5585	0.5815	0.6118	0.6283	0.6527	0.5064	3.305	3.306	3.307	3.308	3.308	0.456
					12 hr 0.1130*	24 hr 0.9500	2.196				12 hr 3.380	24 hr 3.453	2.196
Rebound: 0	0	0.9082			12 hr 0.9001		2.146	3.449				12 hr 3.449	2.148

* Dial ran out during test -- reset reading

TABLE 5

RESULTS OF LOAD TEST ON PILE #4

Location: West End City Yard
114 Ave. & 144th Street
Edmonton, Alberta

Trial #2
Pile: 12" Ø - 23' deep - no bell
No end bearing

Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	Level Readings (ft) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
0	0	0.1418						3.464					
12.5	300	0.1795	0.1796	0.1810	0.1811	0.1812	0.0394	3.466	3.466	3.467		3.467	0.036
25.0	1,190	0.1902	0.1915	0.1931	0.1940	0.1940	0.0522	3.467	3.468	3.468		3.468	0.048
37.5	1,940	0.2095	0.2165	0.2215	0.2245	0.2274	0.0856	3.468	3.468	3.469		3.469	0.060
50	2,700	0.2460	0.2624	0.2741	0.2841	0.2904	0.1486	3.470				3.471	0.084
62.5	3,450	0.3115	0.3622	0.3940	0.4343	0.4669	0.3251	3.472	3.480	3.481	3.485	3.487	0.276
65	3,600	0.4767	0.5180	0.5595	0.6025	0.6495	0.5077	3.487	3.490	3.495	3.500	3.504	0.480
						24 hr 0.4720*	2.244					24 hr 3.651	2.244
Rebound: 0	0	0.3925		12 hr 0.3915			2.164	3.645				12 hr 3.644	2.164

*Dial ran out during test -- reset reading

TABLE 6

RESULTS OF LOAD TEST ON PILE #5

Location: West End City Yard
114th Ave. & 144 St.
Edmonton, Alberta

Trial #1
Pile: 12" Ø - 26' deep
End bearing only

Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	Level Readings (ft) 0.min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
0	0	0.1923						3.417					
12.5	300	0.1290	0.2549	0.2728	0.2839	0.2888		3.531	3.544	3.545	3.546	3.547	1.560*
25	1,190	+						3.642	3.660	3.671	3.678	3.687	3.040*
37.5	1,940							3.820	3.845	3.855	3.862	3.870	5.436*
50	2,700							4.036	4.040	4.042 3 hr 4.053	4.044 11 hr 4.055	4.045 24 hr 4.055	7.536* 7.656*
Rebound: 0	0									4.035			7.416*

+Dial not used for balance of test

*Level readings used in plotting load settlement curve

TABLE 7

RESULTS OF LOAD TEST ON PILE #5

Location: West End City Yard
114th Ave. & 144th St.
Edmonton, Alberta

Trial #2
Pile - 12" Ø - 26' deep
End bearing only

Load (tons)	Reading on Jack	Extensometer Dial (ins)						Level Readings (ft)					
		0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
0	0	0.2355						4.040					
12.5	300	0.2760	0.2880	0.2903	0.2903	0.2903	0.0548	4.045					0.060
25	1,190	0.3860	0.4158	0.4302	0.4379	0.4422	0.2067	4.050	4.053	4.055	4.055	4.055	0.180
37.5	1,940	0.6645	0.7507	0.8022	0.8200	0.8712	0.6357	4.072	4.080	4.084	4.086	4.089	0.588
50	2,700					0.7040+		4.532				4.550	6.110*
			17 1/2 hr 0.8927	24 hr 0.9892					17 1/2 hr 4.565	24 hr 4.570			6.350*
Rebound:													
0	0	0.8382			15 hr 0.8154			4.559				15 hr 4.555	6.180*

+Dial ran out during test -- reset reading

*Level readings used in plotting load settlement curve

TABLE 8

RESULTS OF LOAD TEST ON PILE #5

Location: West End City Yard
114th Ave. & 144th St.
Edmonton, Alberta

Trial #3
Pile: 12" Ø - 26' deep
End bearing only

Load (tons)	Reading on Jack	Extensometer Dial (ins) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	Level Readings (ft) 0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
0	0	0.7272						4.590					
12.5	300	0.7135	0.7098	0.7088	0.7084	0.7072	0.0200	4.592	4.592	4.592	4.592	4.592	0.024
25	1,190	0.6072	0.5988	0.5942	0.5926	0.5892	0.1380	4.600	4.600	4.600	4.601	4.601	0.132
37.5	1,940	0.4988	0.4759	0.4694	0.4656	0.4642	0.2630	4.608	4.609	4.609	4.609	4.609	0.228
50	2,700	0.4027	0.3561	0.3332	0.3150	0.3037	0.4235	4.616	4.619	4.620	4.622	4.623	0.396
55	3,000	0.2610				0.4720+		4.627				4.630 24 hr 4.651	0.480 0.732*
Rebound: 0	0	0.8335				24 hr 0.8690		4.623				24 hr 4.622	0.384*

+Dial ran out during test -- reset reading

*Level readings used in plotting load settlement curves

TABLE 9

RESULTS OF LOAD TEST ON PILE #3

Location: West End City Yard
114th Ave. & 144th St.,
Edmonton, Alberta

Trial #1
Pile: 12" Ø - 26' deep - no bell

Load (tons)	Reading on Jack	Extensometer Dial (ins)						Level Readings (ft)					
		0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)	0 min	15 min	30 min	45 min	1 hr	Total Settl'n (ins)
8.75	0	0.2767						3.187					
17.5	650	0.2769	0.2805	0.2805	0.2829	0.2837	0.0070	3.187				3.187	0
26.25	1,250	0.2892	0.2920	0.2938	0.2955	0.2965	0.0198	3.188				3.188	0.012
35	1,800	0.3020	0.3069	0.3103	0.3119	0.3142	0.0375	3.189				3.190	0.036
43.75	2,300	0.3192	0.3229	0.3240	0.3270	0.3281	0.0514	3.190				3.191	0.048
52.5	2,850	0.3368	0.3437	0.3512	0.3529	0.3538	0.0771	3.192				3.193	0.072
70	3,900	0.3775	1 hr 0.4065	5 hr 0.4530		24 hr 0.5762	1 hr 0.1298 24 hr 0.2995	3.196		1 hr 3.200	5 hr 3.203	24 hr 3.214	1 hr 0.156 24 hr 0.372
Rebound 35	1,800	0.5655					0.2888	3.212					0.300
17.5	650	0.5531					0.2764	3.211					0.288
0	0	0.5371				24 hr 0.4988	0.2221	3.210				24 hr 3.205	0.216

APPENDIX A

LABORATORY TEST RESULTS

SITE OF CHEMISTRY AND PHYSICS BUILDING

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	TEST PILE #2
SITE	CHEM-PHYSICS BUILDING.
SAMPLE	#1
LOCATION	UNIVERSITY CAMPUS
HOLE #2	DEPTH 5'
TECHNICIAN P.K.	DATE

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	41	41	38	14	13	16
Container No.	V79	V46	V20	V71	V41	V36
Wt. Sample Wet + Tare	93.7315	99.5072	82.0329	86.0770	81.7600	82.3058
Wt. Sample Dry + Tare	89.7084	95.0826	77.5077	81.9094	78.1030	77.7132
Wt. Water	4.0231	4.4246	4.5252	4.1676	3.6570	4.5926
Tare Container	79.5047	83.7237	65.5906	72.0433	69.6448	67.0527
Wt. of Dry Soil	10.2037	11.3589	11.9371	9.8661	8.4582	10.6605
Moisture Content w%	39.3	39.0	38.0	42.3	43.2	43.1

Average Values

w_l = 40.6 %

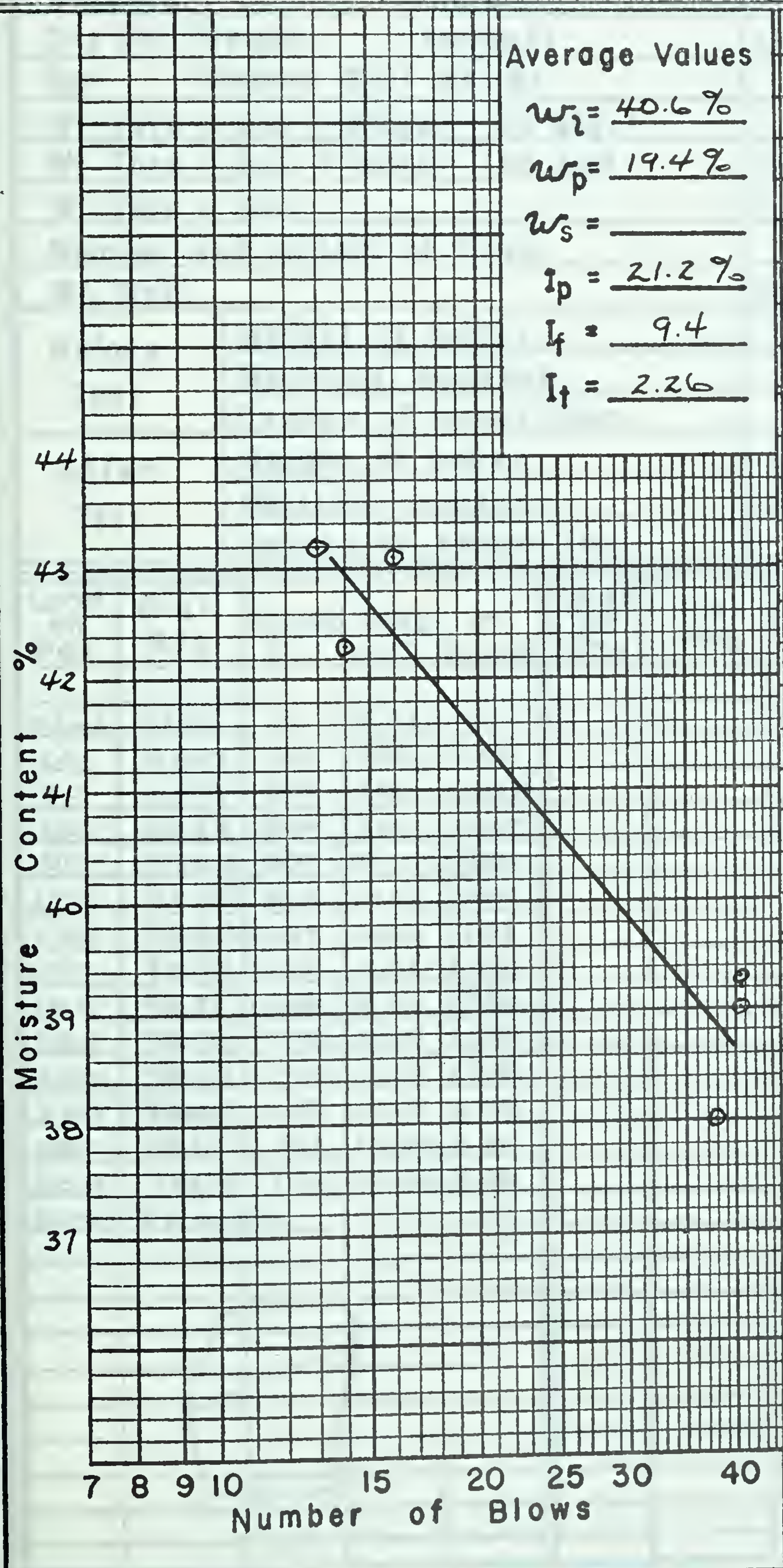
w_p = 19.4 %

w_s = ______

I_p = 21.2 %

I_f = 9.4

I_t = 2.26



Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	32.4303	32.8045	42.3165
Wt. Sample Dry + Tare	32.0235	32.3281	41.7983
Wt. Water	0.4068	0.4764	0.5182
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	2.1332	2.4267	2.6921
Moisture Content %	19.2	19.6	19.3

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V - V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V - V_o}{W_o} \times 100\right)$$

Description of Sample: **CLAY**:- firm, medium plastic, light brown, rust lenses present.

Remarks:

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #2
SITE Chem-Physics Building.
SAMPLE #1
LOCATION University Campus.
HOLE #2 DEPTH 5'
TECHNICIAN P.K. DATE 1/14/59

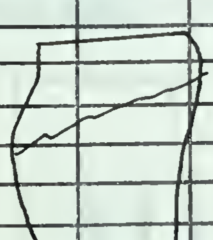
Machine Data:- Unconfined Test
Machine No. 2
Multiplication Factor x 100
Wt. Loading Block + Piston (gms.)

Description of Sample: Clay:- firm, medium plastic, light brown, rust lenses present.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure (σ_3)								
Length		inches	3.14					
Area Diam = 1.4"		sq. cms.	9.93					
Volume		c. c. s.	79.1					
Dry Unit Weight		lbs/cu. ft.	109.5					
Gs= ·Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			173.94					
Wt. Tare + Soil								
Number and weight of Tare								
Wt. Soil			138.94					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		35.00					
	Moisture content		25.2 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gms	.0216	0	9.93	0										
20	.0169	.150	9.94	0.201										
40	.0109	.340	9.96	0.402										
60	.0059	.500	9.98	0.602										
80	.9965	.800	10.01	0.800										
100	.9945	.863	10.02	1.000										
120	.9885	1.053	10.03	1.193										
140	.9878	1.078	10.03	1.393										
160	.9872	1.098	10.03	1.590										
180	.9870	1.102	10.04	1.791										
200	.9869	1.108	10.04	1.990										
220	.9865	1.118	10.04	2.190										
280	.9861	1.131	10.05	2.8										
300	.9860	1.135	10.05	3.0										
320	FAILED													



UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #2
SITE Chem - Physics Building.
SAMPLE #2
LOCATION University Campus.
HOLE #2 DEPTH 10'
TECHNICIAN P.K. DATE 1/9/59

Machine Data:- Unconfined Test.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: Glacial Till :- weathered, pockets of coal present, moist, non plastic, dense.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure	(σ_3)						
Length	inches	4.4					
Area Diam 1.8"	sq. cms.	16.4					
Volume	c.c.s.	183.2					
Dry Unit Weight	lbs/cu.ft.	121.5					
G_S= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		440.41					
Wt. Tare + Soil		387.99					
Number and weight of Tare		30.73					
Wt. Soil		357.26					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water						
	Moisture content						
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm²	σ_1 Kgm/cm²
0	.5628	0	16.4	0
100	.5535	.211	16.45	.304
200	.5418	.476	16.48	.607
300	.5324	.690	16.52	.908
400	.5309	.724	16.55	1.21
450	.5295	=.8938		1.36
500	.8908	.824	16.56	1.51
600	.8850	.955	16.58	1.81
700	.8717	1.26	16.62	2.10
800	.8655	1.40	16.65	2.40
900	.8515	1.716	16.69	2.80
1000	.8340	2.110	16.78	2.98
1100	.8270	2.275	16.80	3.27
1200	.8210	2.282	16.80	3.57
1300	.7595	2.420	16.80	3.87
1400	.7085	4.97	17.26	4.06
1500	.6868	5.46	17.35	4.32
1600	FAILED.			

Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
		.766	16.56	1.36					

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

MOISTURE CONTENT

PROJECT Test Pile #2
SITE Chem-Physics Building
SAMPLE
LOCATION University Campus
HOLE DEPTH
TECHNICIAN P.K. DATE 11/22/58

Hole No.	2	2	2	2	2	2
Depth	2½'	5'	7½'	10'	12½'	15'
Sample No.		1		2		3
Container No.	1A130	1A27	1A27	1A27	1A35	1A27
Wt. Sample Wet + Tare	87.95	71.36	97.34	67.81	100.64	63.00
Wt. Sample Dry + Tare	71.25	60.36	85.76	62.63	88.83	58.81
Wt. Water	16.69	11.00	11.58	5.18	11.81	4.09
Tare Container	17.54	17.67	17.67	17.66	17.71	17.66
Wt. of Dry Soil	53.71	42.69	68.09	44.97	71.12	41.15
Moisture Content w %	31.2	25.8	16.9	11.6	16.6	10.0
Hole No	2	2	2			
Depth	17½'	20'	23'			
Sample No.		4				
Container No.	1A19	1A86	1A23			
Wt. Sample Wet + Tare	85.69	67.12	74.41			
Wt. Sample Dry + Tare	77.90	62.29	64.33			
Wt. Water	7.79	4.83	10.08			
Tare Container	17.46	17.38	17.27			
Wt. of Dry Soil	60.44	44.91	47.06			
Moisture Content w %	12.9	10.8	22.8			
Hole No.						
Depth						
Sample No.						
Container No.						
Wt. Sample Wet + Tare						
Wt. Sample Dry + Tare						
Wt. Water						
Tare Container						
Wt. of Dry Soil						
Moisture Content w%						

Remarks:

APPENDIX B

LABORATORY TEST RESULTS

SITE OF WEST-END CITY YARDS

LABORATORY TEST RESULTS

TEST PILE NO. 3

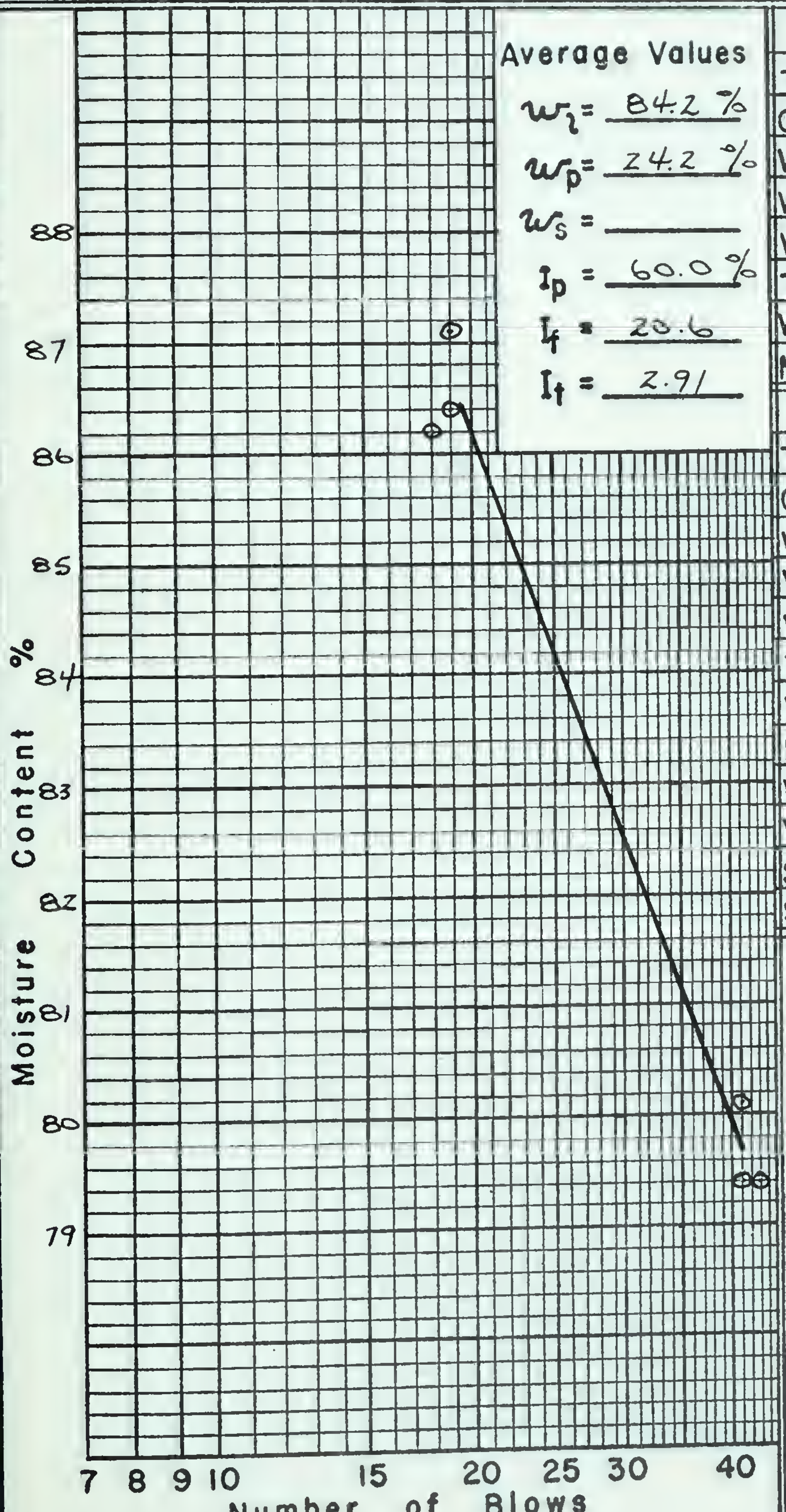
UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

PROJECT	Test Pile #3
SITE	114th Ave. & 144th Street.
SAMPLE	#1
LOCATION	
HOLE #3	DEPTH 5'
TECHNICIAN P.K.	DATE 24/1/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	41	41	43	18	19	19
Container No.	V41	V36	V79	V71	V46	V20
Wt. Sample Wet + Tare	84.2053	81.7601	92.6428	88.8034	100.4097	80.6532
Wt. Sample Dry + Tare	77.7311	75.2533	86.8244	81.0517	92.6834	73.6767
Wt. Water	6.4742	6.5068	5.8184	7.7517	7.7263	6.9765
Tare Container	69.6448	67.0527	79.5047	72.0433	83.7237	65.5906
Wt. of Dry Soil	8.0863	8.2006	7.3167	9.0084	8.8597	8.0861
Moisture Content w%	80.1	79.4	79.4	86.2	87.1	86.4



Average Values

w_l = 84.2 %

w_p = 24.2 %

w_s = ______

I_p = 60.0 %

I_f = 20.6

I_t = 2.91

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet+Tare	31.7343	31.7412	40.6491
Wt. Sample Dry+Tare	31.3717	31.3844	40.3068
Wt. Water	0.3626	0.3568	0.3423
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	1.4814	1.4830	1.4021
Moisture Content %	24.5	24.1	24.4

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
CLAY:- highly plastic, moist, firm, nugget structure present.

Remarks:

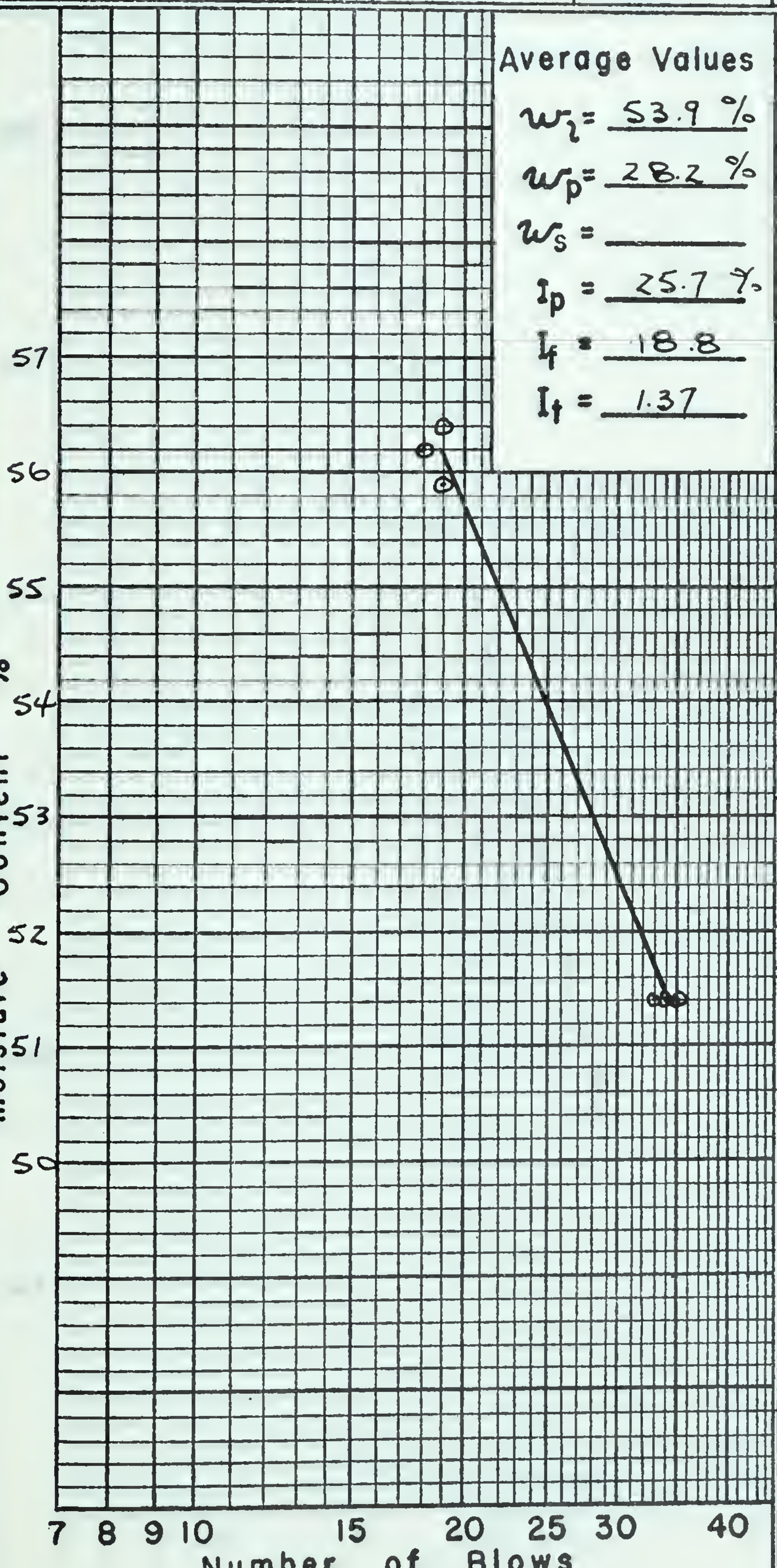
UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

PROJECT TEST PILE #3
SITE
SAMPLE #2
LOCATION 144nd ST. & 114th AVE.
HOLE #3 DEPTH 10'
TECHNICIAN P.K. DATE 1/18/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	35	34	33	19	19	18
Container No.	V79	V20	V46	V41	V71	V36
Wt. Sample Wet + Tare	95.3145	81.3697	98.2450	83.2292	89.7113	81.9485
Wt. Sample Dry + Tare	89.9376	76.0099	93.3234	78.3354	83.3685	76.5907
Wt. Water	5.3769	5.3598	4.9216	4.8938	6.3428	5.3578
Tare Container	79.5047	65.5906	83.7237	69.6448	72.0433	67.0527
Wt. of Dry Soil	10.4329	10.4193	9.5997	8.6906	11.3252	9.5380
Moisture Content w%	51.4	51.4	51.4	56.4	55.9	56.2



Average Values

w_l = 53.9 %
w_p = 28.2 %
w_s =
I_p = 25.7 %
I_f = 18.8
I_t = 1.37

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	32.2570	32.5477	41.6348
Wt. Sample Dry + Tare	31.7307	31.9572	41.0900
Wt. Water	0.5263	0.5905	0.5448
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	1.8404	2.0558	1.9838
Moisture Content %	28.6	28.7	27.4

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: **CLAY**:- medium plastic, firm, nugget structure present, mottled grey & brown.

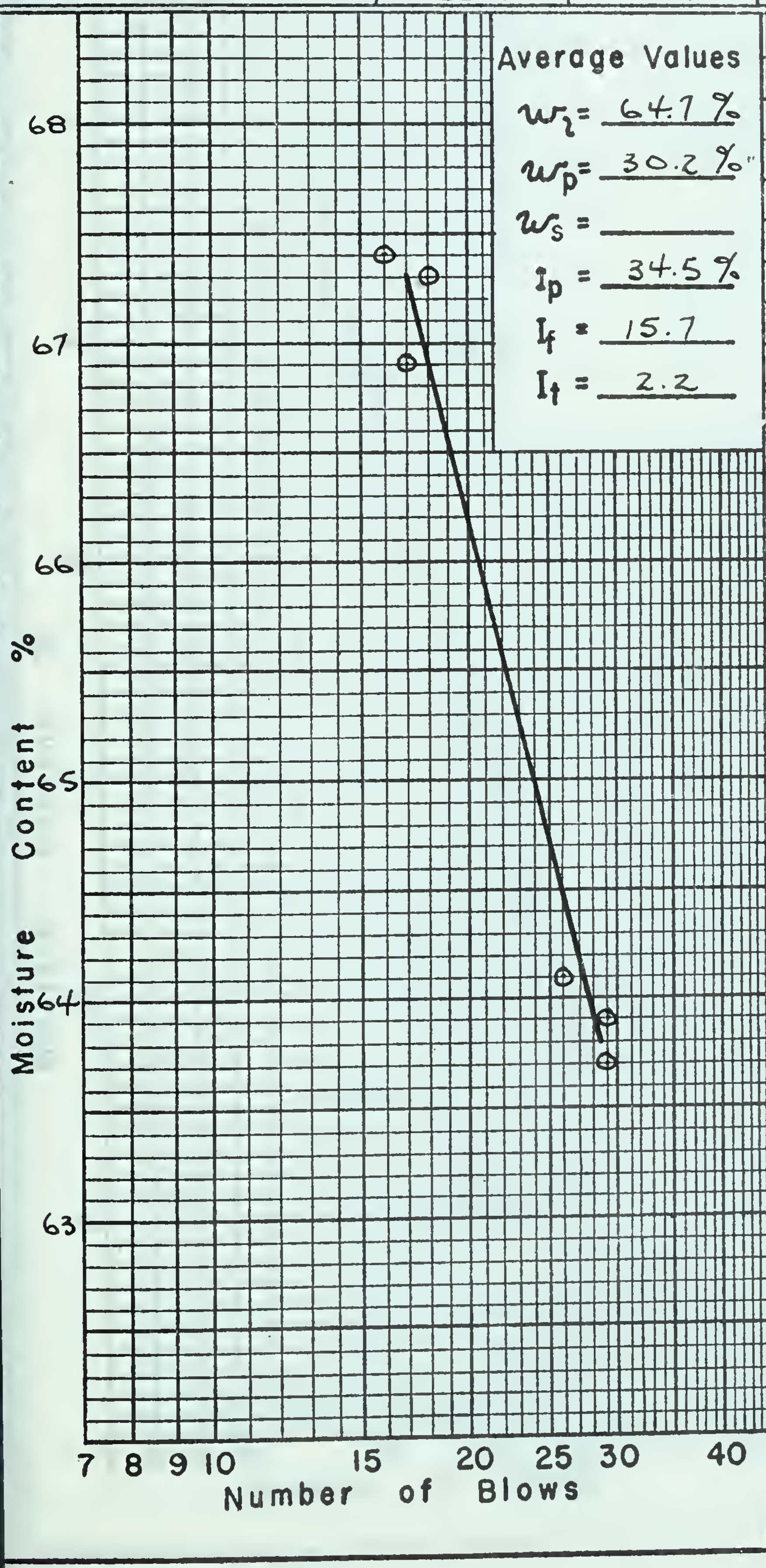
Remarks:

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	TEST PILE #3
SITE	114th AVE. & 144th STREET.
SAMPLE	#3
LOCATION	
HOLE	#3 — DEPTH 15'.
TECHNICIAN	P. K. — DATE 1/18/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	29	26	29	16	17	18
Container No.	V41	V79	V36	V20	V71	V46
Wt. Sample Wet + Tare	86.6003	96.4950	83.6643	83.0263	89.9570	99.9474
Wt. Sample Dry + Tare	79.9860	89.8516	77.1985	75.9916	82.7767	93.4211
Wt. Water	6.6143	6.6434	6.4658	7.0347	7.1804	6.5263
Tare Container	69.6448	79.5047	67.0527	65.5706	72.0433	83.7237
Wt. of Dry Soil	10.3412	10.3469	10.1458	10.4210	10.7334	9.6974
Moisture Content $w\%$	63.9	64.1	63.7	67.4	66.9	67.3



Average Values

w_l = 64.7 %

w_p = 30.2 %

w_s =

I_p = 34.5 %

I_f = 15.7

I_t = 2.2

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet+Tare	33.8988	33.3584	42.0207
Wt. Sample Dry + Tare	32.9590	32.5561	41.3512
Wt. Water	0.9398	0.8023	0.6695
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.0687	2.6547	2.2450
Moisture Content %	30.6	30.2	29.8

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content $w\%$			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:

CLAY:- grey, firm, highly plastic, moist.

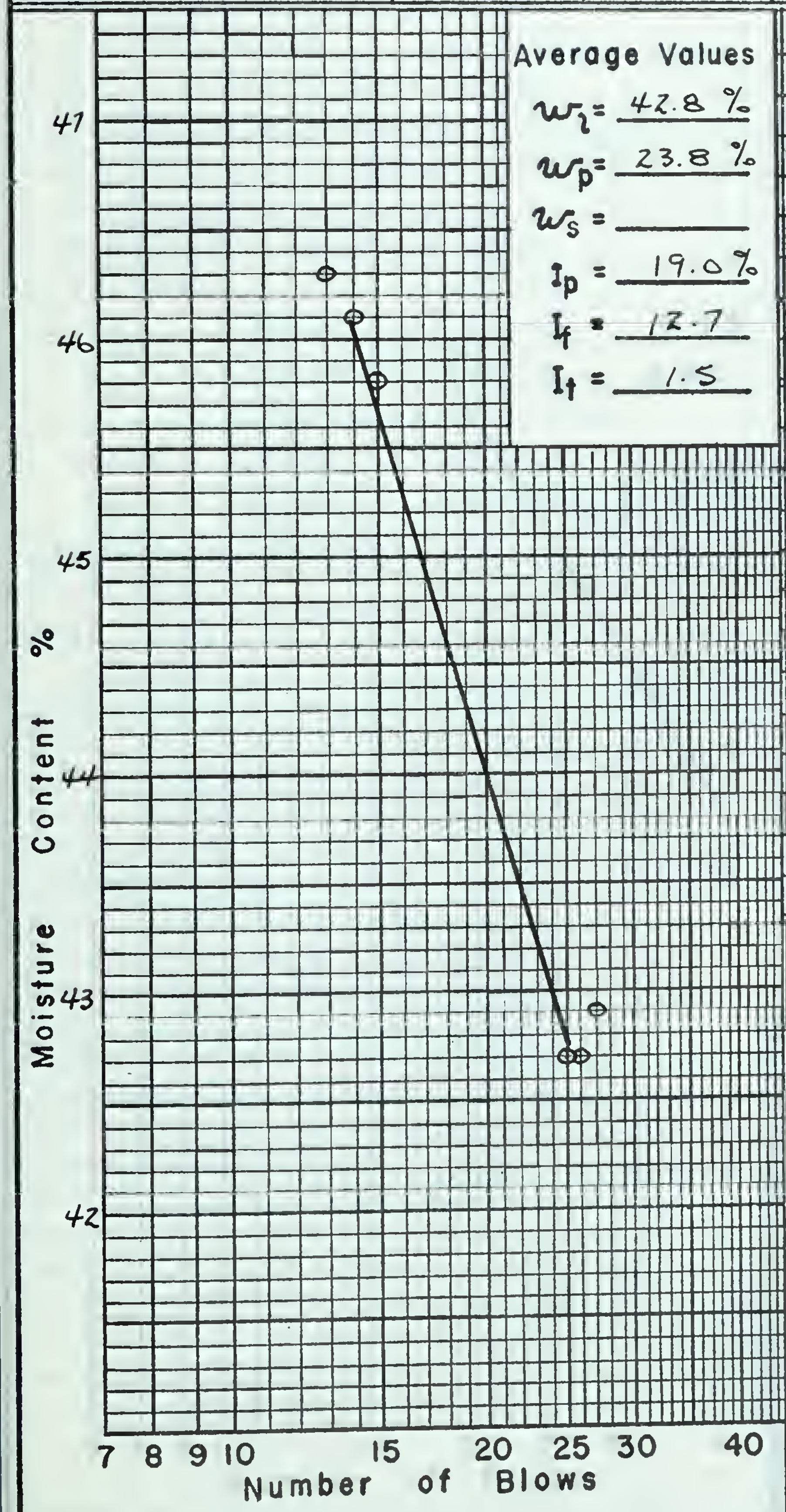
Remarks:

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #3
SITE 114th Ave & 144th Street.
SAMPLE #4
LOCATION
HOLE #3 DEPTH 20'
TECHNICIAN P.K. DATE 1/19/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	27	26	25	13	14	15
Container No.	V36	V41	V20	V46	V79	V71
Wt. Sample Wet + Tare	87.1472	86.8683	85.7276	102.6605	94.8541	89.2773
Wt. Sample Dry + Tare	81.1149	81.7073	79.6938	96.6626	90.0062	83.8578
Wt. Water	6.0323	5.1610	6.0338	5.9979	4.8479	5.4195
Tare Container	67.0527	69.6448	65.5706	83.7237	79.5047	72.0433
Wt. of Dry Soil	14.0622	12.0625	14.1232	12.9389	10.5015	11.8145
Moisture Content w%	42.9	42.7	42.7	46.3	46.1	45.8



Average Values

w_l = 42.8 %

w_p = 23.8 %

w_s =

I_p = 19.0 %

I_f = 12.7

I_t = 1.5

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet+Tare	34.3577	33.9256	43.2248
Wt. Sample Dry+Tare	33.4953	33.1574	42.4278
Wt. Water	0.8624	0.7682	0.7970
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.6050	3.2560	3.3216
Moisture Content %	23.9	23.6	24.0

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: CLAY:- firm, grey, medium plastic, moist.

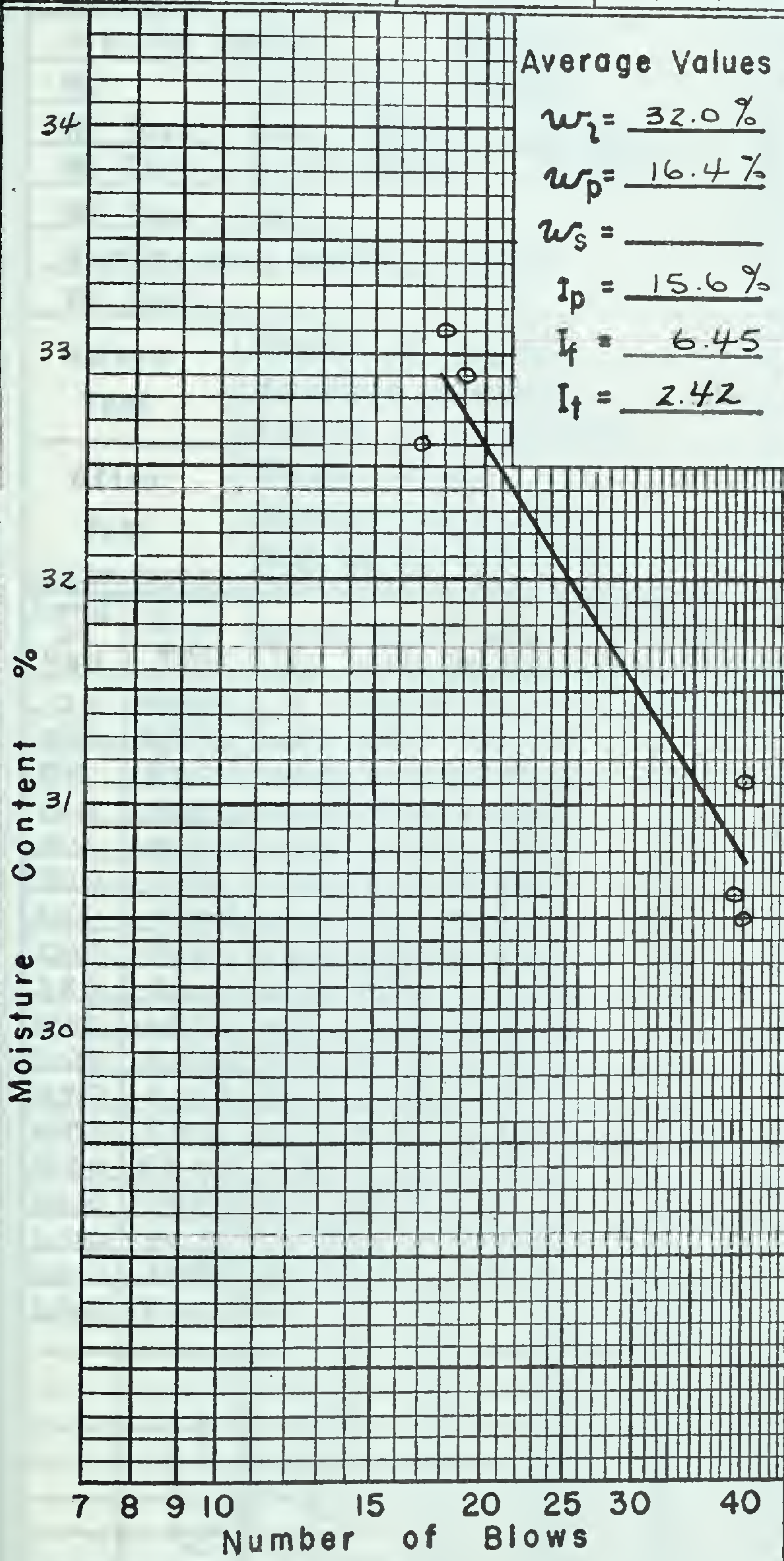
Remarks:

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #3
SITE 114th Ave & 144th Street.
SAMPLE #5
LOCATION
HOLE #3 DEPTH 25'
TECHNICIAN P.K. DATE 1/20/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	39	40	40	17	18	19
Container No.	V71	V41	V36	V79	V20	V46
Wt. Sample Wet + Tare	93.3089	93.7733	84.9006	97.3150	84.1002	100.7605
Wt. Sample Dry + Tare	88.3375	88.1272	80.6900	92.9273	79.4743	96.5500
Wt. Water	4.9714	5.6461	4.2106	4.3877	4.6259	4.2105
Tare Container	72.0433	69.6448	67.0527	79.5047	65.5706	83.7237
Wt. of Dry Soil	16.2942	18.4824	13.5373	13.4226	13.9037	12.8263
Moisture Content w%	30.6	30.5	31.1	32.6	33.1	32.8



Average Values

w_l = 32.0 %

w_p = 16.4 %

w_s =

I_p = 15.6 %

I_f = 6.45

I_t = 2.42

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	33.9126	33.2026	42.9397
Wt. Sample Dry + Tare	33.3373	32.7404	42.4013
Wt. Water	0.5753	0.4602	0.5384
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.4470	2.8390	3.2951
Moisture Content %	16.7	16.2	16.4

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
GLACIAL TILL:- dense, grey, sandy, coal & pea gravel present, medium plastic.

Remarks:

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SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT Test Pile #3
SITE 114th Ave. & 144th Street.
SAMPLE #1
LOCATION
HOLE #3 DEPTH 5'
TECHNICIAN P.K. DATE 1/16/59

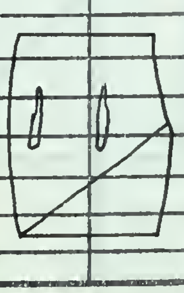
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- highly plastic, moist, firm, nugget structure present

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.3					
Area Diam. = 1.7"		sq. cms.	14.4					
Volume		c.c.s.	157					
Dry Unit Weight		lbs/cu.ft.	113.0					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			404.0					
Wt. Tare + Soil			306.0					
Number and weight of Tare			20.0					
Wt. Soil			286.0					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		98.0					
	Moisture content		34.3 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cms.	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 g	.8638	0	14.4	0										
40	.8592	0.105	14.4	0.139										
80	.8504	0.305	14.45	0.278										
120	.8356	0.642	14.50	0.414										
160	.8175	1.03	14.56	0.550										
200	.7961	1.52	14.63	0.682										
240	.7749	2.02	14.71	0.815										
280	.7482	2.63	14.80	0.945										
320	.7212	3.24	14.90	1.073										
360	.6892	3.95	14.99	1.201										
400	.6524	4.81	15.13	1.321										
440	.6162	5.64	15.28	1.440										
480	.5813	6.44	15.40	1.558										
520	.5509	7.28	15.53	1.672										
560	.4882	8.74	15.79	1.77										
600	.4175	10.4	16.1	1.86										
640	.2889	13.18	16.6	1.93										
660	FAILED													

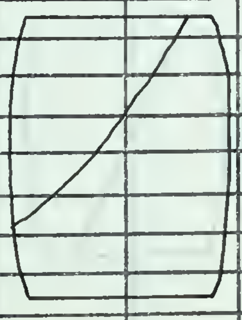


UNIVERSITY of ALBERTA DEPT. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY TRI-AXIAL COMPRESSION	PROJECT TEST PILE #3 SITE 114th AVE & 144th STREET SAMPLE #2 LOCATION HOLE #3 DEPTH 10' TECHNICIAN P.K. DATE 1/18/59
Machine Data:- UNCONFINED TEST. Machine No. 9 Multiplication Factor x 50 Wt. Loading Block + Piston (gms.)	Description of Sample: CLAY:- medium plastic, firm, nugget structure present, mottled grey & brown.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	3.15					
Area Diam = 1.4"		sq. cms.	9.93					
Volume		c.c.s.	79.4					
Dry Unit Weight		lbs/cu.ft.	93.0					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			191.34					
Wt. Tare + Soil			149.25					
Number and weight of Tare			30.73					
Wt. Soil			118.52					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		42.09					
	Moisture content		35.5 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm²	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gms	.7315	0	9.93	0										
20	.7290	0.008	9.93	0.101										
40	.7262	0.017	9.93	0.202										
50	.7250	0.021	9.93	0.252										
100	.7179	0.043	9.93	0.504										
150	.7025	0.092	9.93	0.755										
200	.6852	0.147	9.94	1.01										
250	.6500	0.575	9.98	1.26										
300	.6376	2.98	10.20	1.51										
350	.5995	4.18	10.35	1.69										
370	.5550	5.61	10.51	1.76										
390	FAILED													



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SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT TEST PILE #3
SITE 114th AVE & 144th STREET.
SAMPLE #3
LOCATION
HOLE #3 DEPTH 15'
TECHNICIAN P.K. DATE 1/18/59

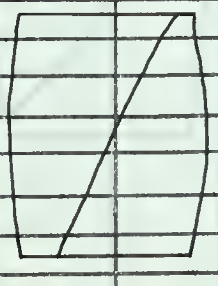
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- grey, firm, high plastic, moist.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	3.6					
Area Diam = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	150.0					
Dry Unit Weight		lbs/cu.ft.	87.8					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			320.65					
Wt. Tare + Soil			244.13					
Number and weight of Tare			30.73					
Wt. Soil			213.40					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		76.52					
	Moisture content		35.9 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm²	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gms	.8200	0	16.4	0										
50	.8096	0.290	16.47	0.152										
100	.7910	0.810	16.53	0.302										
150	.7700	1.39	16.63	0.451										
200	.7505	1.93	16.71	0.598										
250	.7304	2.49	16.89	0.741										
300	.7092	3.00	16.90	0.888										
350	.6880	3.67	17.01	1.03										
400	.6650	4.31	17.15	1.17										
450	.6408	4.97	17.3	1.30										
500	.6020	6.05	17.48	1.43										
550	FAILED.													



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TRI-AXIAL COMPRESSION

PROJECT Test Pile #3
SITE 114th Ave & 144th Street.
SAMPLE #4
LOCATION
HOLE #3 DEPTH 20'
TECHNICIAN P.K. DATE 1/18/59

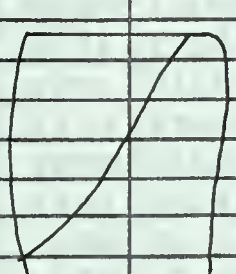
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- stiff, highly plastic, grey, moist,

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.0					
Area Diam. = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	166.5					
Dry Unit Weight		lbs/cu.ft.	85.0					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			339.18					
Wt. Tare + Soil			258.03					
Number and weight of Tare			30.74					
Wt. Soil			227.29					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		81.15					
	Moisture content		35.7%					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm²	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.7642	0	16.4	0										
50	.6843	2.00	16.73	0.15										
100	.6053	3.97	17.08	0.29										
150	.5375	5.67	17.40	0.43										
200	.4622	7.55	17.78	0.56										
250	.3721	9.82	18.2	0.69										
300	.1400	15.6	19.4	0.77										
350	FAILED.													



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SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT Test Pile #3
SITE 114th Ave. & 144th Street.
SAMPLE #5
LOCATION
HOLE #3 DEPTH 25'
TECHNICIAN D.K. DATE 1/18/59

Machine Data:- Unconfined Test.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: Glacial Till:- dense, grey, sandy, coal & pea gravel present, medium plastic.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure	(σ_{3})						
Length	inches	4.75					
Area Diam. = 1.8"	sq. cms.	16.4					
Volume	c.c.s.	197.9					
Dry Unit Weight	lbs/cu.ft.	111.54					
Gs= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		675.99					
Wt. Tare + Soil		615.82					
Number and weight of Tare		249.92					
Wt. Soil		365.90					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	60.17					
	Moisture content	16.4%					
	Degree of saturation						

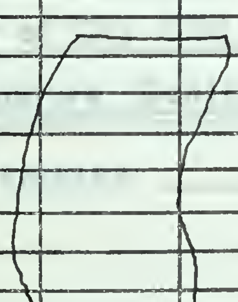
Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gms	.7871	0	16.4	0										
50	.7719	0.320	16.48	0.152										
100	.7492	0.797	16.55	0.302										
150	.7213	1.387	16.63	0.450										
200	.6841	2.170	16.78	0.596										
250	.6475	2.940	16.91	0.739										
300	.6115	3.700	17.05	0.880										
350	.5725	4.530	17.19	1.019										
400	.5345	5.320	17.35	1.152										
450	.4932	6.180	17.49	1.287										
500	.4550	7.000	17.63	1.418										
550	.4191	7.750	17.80	1.544										
600	.3791	8.610	17.97	1.670										
650	.3341	9.550	18.15	1.790										
700	.2878	10.52	18.35	1.909										
750	.2321	11.69	18.59	2.02										
800	.1640	13.12	18.88	2.12										
850	.1435	13.53	18.97	2.24										
900	.0650	15.2	19.32	2.33										
950	FAILED													

UNIVERSITY of ALBERTA DEPT. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **TRI-AXIAL COMPRESSION**	PROJECT Test Pile #3 SITE 114th Ave & 144th Street SAMPLE #6 LOCATION HOLE #3 DEPTH 30' TECHNICIAN P.K. DATE 3/30/59
Machine Data:- Machine No. 5 Multiplication Factor x 100 Wt. Loading Block + Piston (gms.)	Description of Sample: Glacial Till:- silty, low to medium plastic, moist, dense, coal & pea gravel present.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.6					
Area D = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	191.5					
Dry Unit Weight		lbs/cu.ft.	131.0					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			492.63					
Wt. Tare + Soil			432.80					
Number and weight of Tare			30.70					
Wt. Soil			402.10					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		59.83					
	Moisture content		6.7 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0	.8870		16.4											
50	.8600													
100	.8215													
150	.7882													
200	.7532													
250	.7031													
300	.6548													
350	.5605													
400	.4415													
450	.2875	13.0	18.9	2.38										
500	.0100	19.1	20.3	2.46										
550	FAILED													



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SOIL MECHANICS LABORATORY

MOISTURE CONTENT

PROJECT Test Pile #3
SITE 114th Ave & 144th Street.
SAMPLE
LOCATION
HOLE DEPTH
TECHNICIAN P.K. DATE 1/10/59

Hole No.	3	3	3	3	3	3
Depth	2½'	5'	7½'	10'	12½'	15'
Sample No.		1		2		3
Container No.	1A27	1A17	1A53	1A17	1A17	1A27
Wt. Sample Wet + Tare	68.76	75.30	85.65	81.08	109.11	65.83
Wt. Sample Dry + Tare	55.36	61.10	67.52	64.05	84.32	52.61
Wt. Water	13.40	14.20	18.13	17.03	24.79	13.22
Tare Container	17.69	17.43	17.56	17.43	17.43	17.66
Wt. of Dry Soil	37.67	43.67	49.96	46.62	66.89	34.95
Moisture Content w %	35.6	32.6	36.4	39.6	37.0	38.8

Hole No	3	3	3	3	3	3
Depth	17½'	20'	22½'	25'	26'	27'
Sample No.		4		5		
Container No.	1A23	1A27	1A35	1A27	1A30	7B
Wt. Sample Wet + Tare	104.88	85.59	102.33	75.57	115.34	100.92
Wt. Sample Dry + Tare	80.19	70.15	79.06	66.91	101.01	90.55
Wt. Water	24.69	19.44	23.27	8.66	14.33	10.37
Tare Container	17.27	17.66	17.71	17.66	17.56	24.52
Wt. of Dry Soil	62.92	52.49	61.35	49.25	83.45	66.03
Moisture Content w %	39.3	37.1	38.0	17.6	17.2	15.1

Hole No.	3					
Depth	30'					
Sample No.	6					
Container No.	7C					
Wt. Sample Wet + Tare	101.89					
Wt. Sample Dry + Tare	92.40					
Wt. Water	9.49					
Tare Container	24.52					
Wt. of Dry Soil	67.88					
Moisture Content w%	14.0					

Remarks:

LABORATORY TEST RESULTS

TEST PILE NO. 4

UNIVERSITY of ALBERTA
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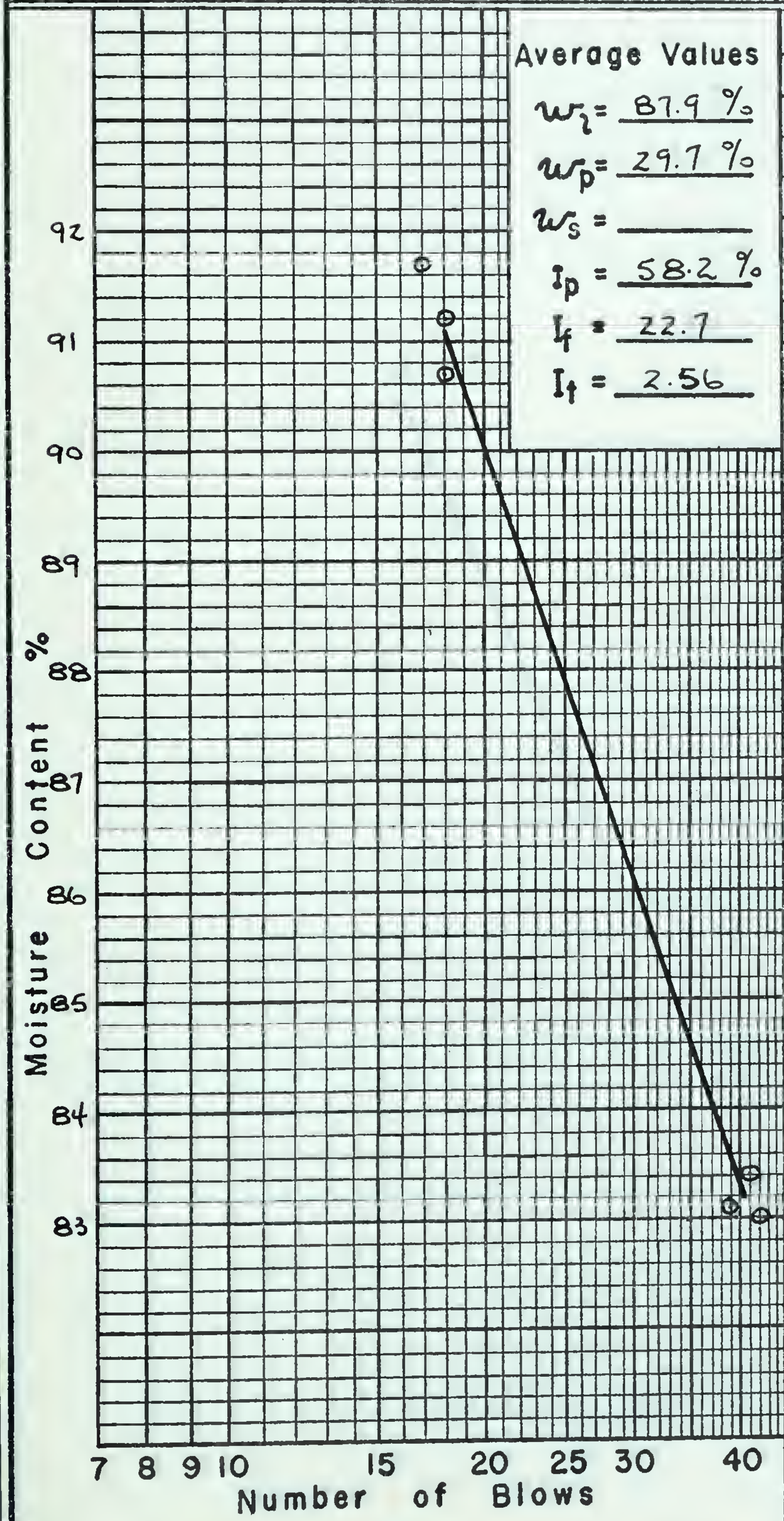
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	Test Pile #4
SITE	114th Ave & 144th Street.
SAMPLE	#1
LOCATION	
HOLE #4	DEPTH 3'
TECHNICIAN P.K.	DATE 1/29/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	41	39	42	18	18	17
Container No.	V71	V46	V36	V41	V79	V20
Wt. Sample Wet + Tare	86.7321	97.9521	84.0857	84.0935	94.3140	81.1706
Wt. Sample Dry + Tare	80.0564	91.4993	76.3605	77.2037	87.2707	73.7135
Wt. Water	6.6757	6.4528	7.7252	6.8898	7.0433	7.4571
Tare Container	72.0433	83.7237	67.0527	69.6448	79.5047	65.5906
Wt. of Dry Soil	8.0131	7.7756	9.3078	7.5589	7.7660	8.1229
Moisture Content w%	83.4	83.1	83.0	91.2	90.7	91.7



Average Values

w_l = 87.9 %

w_p = 29.7 %

w_s =

I_p = 58.2 %

I_f = 22.7

I_t = 2.56

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	34.0925	32.6441	42.2254
Wt. Sample Dry + Tare	33.1327	32.0180	41.5078
Wt. Water	0.9598	0.6261	0.7176
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.2424	2.1166	2.4016
Moisture Content %	29.6	29.7	29.9

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

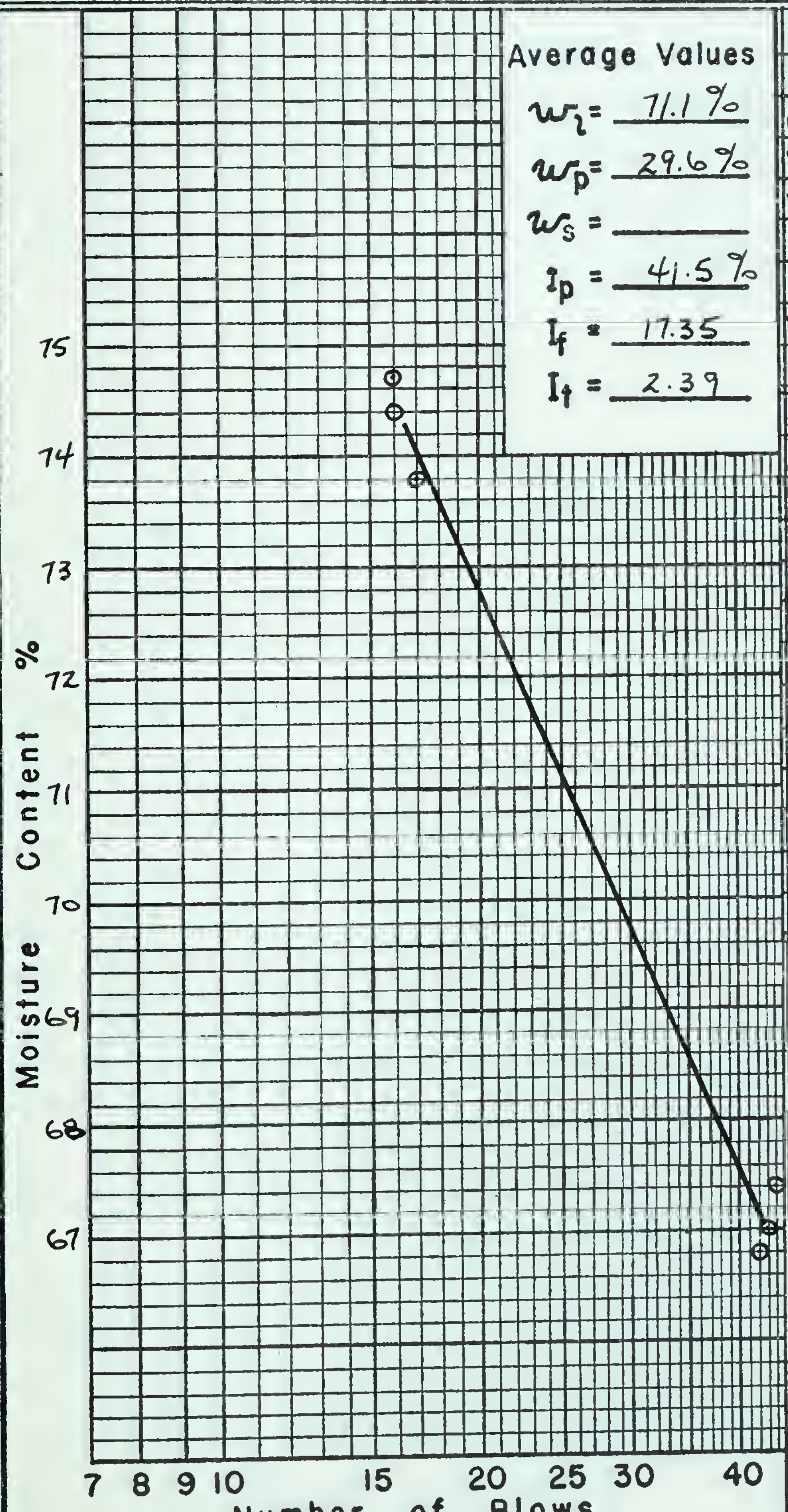
Description of Sample:
Clay:- highly plastic, moist, firm, grey, nugget structure present.

Remarks:

UNIVERSITY of ALBERTA DEP'T. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY ATTERBERG LIMITS	
PROJECT Test Pile #4	
SITE 114th Ave & 144th Street.	
SAMPLE #2	
LOCATION	
HOLE #4	DEPTH 8'
TECHNICIAN P.K.	DATE 1/29/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	44	43	42	16	17	16
Container No.	1	2	A4	A12	V50	V70
Wt. Sample Wet + Tare	88.6723	91.6480	94.5925	88.6320	98.4863	97.5475
Wt. Sample Dry + Tare	82.7741	84.9544	87.3712	81.2685	91.2905	90.9651
Wt. Water	5.8982	6.6936	7.2213	7.3635	7.1958	6.5824
Tare Container	74.0352	74.9626	76.5604	71.3514	81.5525	82.1483
Wt. of Dry Soil	8.7389	9.9918	10.8108	9.9171	9.7380	8.8168
Moisture Content w%	67.4	67.0	66.8	74.4	73.8	74.7



Average Values

w_l = 71.1 %

w_p = 29.6 %

w_s =

I_p = 41.5 %

I_f = 17.35

I_t = 2.39

Plastic Limit

Trial No.	1	2	3
Container No.	1	2	3
Wt. Sample Wet + Tare	51.6249	52.0065	47.6304
Wt. Sample Dry + Tare	50.6001	50.8155	46.7544
Wt. Water	1.0248	1.1910	0.8760
Tare Container	47.1130	46.7891	43.8264
Wt. of Dry Soil	3.4871	4.0264	2.9280
Moisture Content %	29.4	29.6	29.9

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: CLAY:- highly plastic, moist, firm, nugget structure present, brownish grey in color.

Remarks:

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SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

PROJECT Test Pile #4	
SITE 114th Ave & 144th Street.	
SAMPLE #3	
LOCATION	
HOLE #4	DEPTH 13'
TECHNICIAN P.K.	DATE 1/30/59

Liquid Limit

Trial No.	1	2	3			
No. of Blows	24	24	26			
Container No.	V20	V71	V79			
Wt. Sample Wet + Tare	81.3943	87.1538	96.3688			
Wt. Sample Dry + Tare	75.1911	81.2461	89.7317			
Wt. Water	6.2032	5.9077	6.6371			
Tare Container	65.5906	72.0433	79.5047			
Wt. of Dry Soil	9.6005	9.2028	10.2270			
Moisture Content w%	64.6	64.2	64.8			

Moisture Content %

7 8 9 10 15 20 25 30 40

Number of Blows

Average Values

w_l = 64.5 %

w_p = 30.1 %

w_s =

I_p = 34.4 %

I_f =

I_t =

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet+Tare	33.3552	34.1638	44.4300
Wt. Sample Dry+Tare	32.5482	33.1760	43.1280
Wt. Water	0.8070	0.9878	1.2020
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	2.6579	3.2746	4.0218
Moisture Content %	30.4	30.2	29.8

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: Clay:- highly plastic, moist, firm, grey in color.

Remarks:

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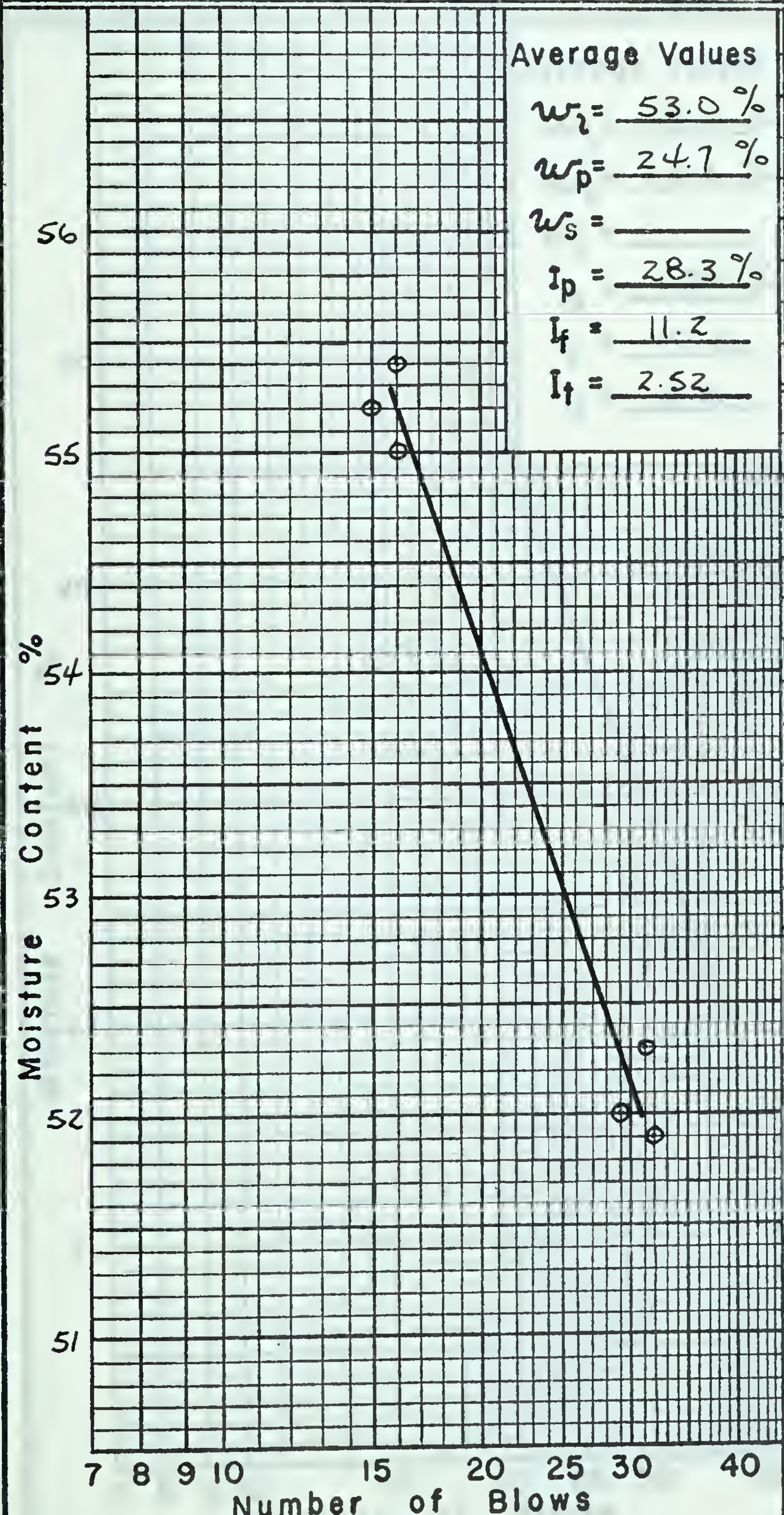
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #4
SITE 114th Ave & 144th Street
SAMPLE #4
LOCATION
HOLE #4 DEPTH 18'
TECHNICIAN P.K. DATE 2/1/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	32	31	29	15	16	16
Container No.	V36	V71	V46	V41	V79	V20
Wt. Sample Wet + Tare	82.1767	89.7555	101.9069	85.1794	98.7343	82.5908
Wt. Sample Dry + Tare	77.0070	83.6811	95.6794	79.6592	91.8805	76.5677
Wt. Water	5.1697	6.0744	6.2275	5.5202	6.8538	6.0231
Tare Container	67.0527	72.0433	83.7237	69.6448	79.5047	65.5906
Wt. of Dry Soil	9.9543	11.6378	11.9557	10.0144	12.3758	10.9771
Moisture Content w%	51.9	52.3	52.0	55.2	55.4	55.0



Average Values

w_l = 53.0 %
w_p = 24.7 %
w_s =
I_p = 28.3 %
I_f = 11.2
I_t = 2.52

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	33.7439	33.1221	41.8943
Wt. Sample Dry + Tare	32.9763	32.4760	41.3441
Wt. Water	0.7676	0.6461	0.5502
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.0860	2.5746	2.2379
Moisture Content %	24.8	24.7	24.6

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
CLAY:- highly plastic, stiff, moist, grey in color.

Remarks:

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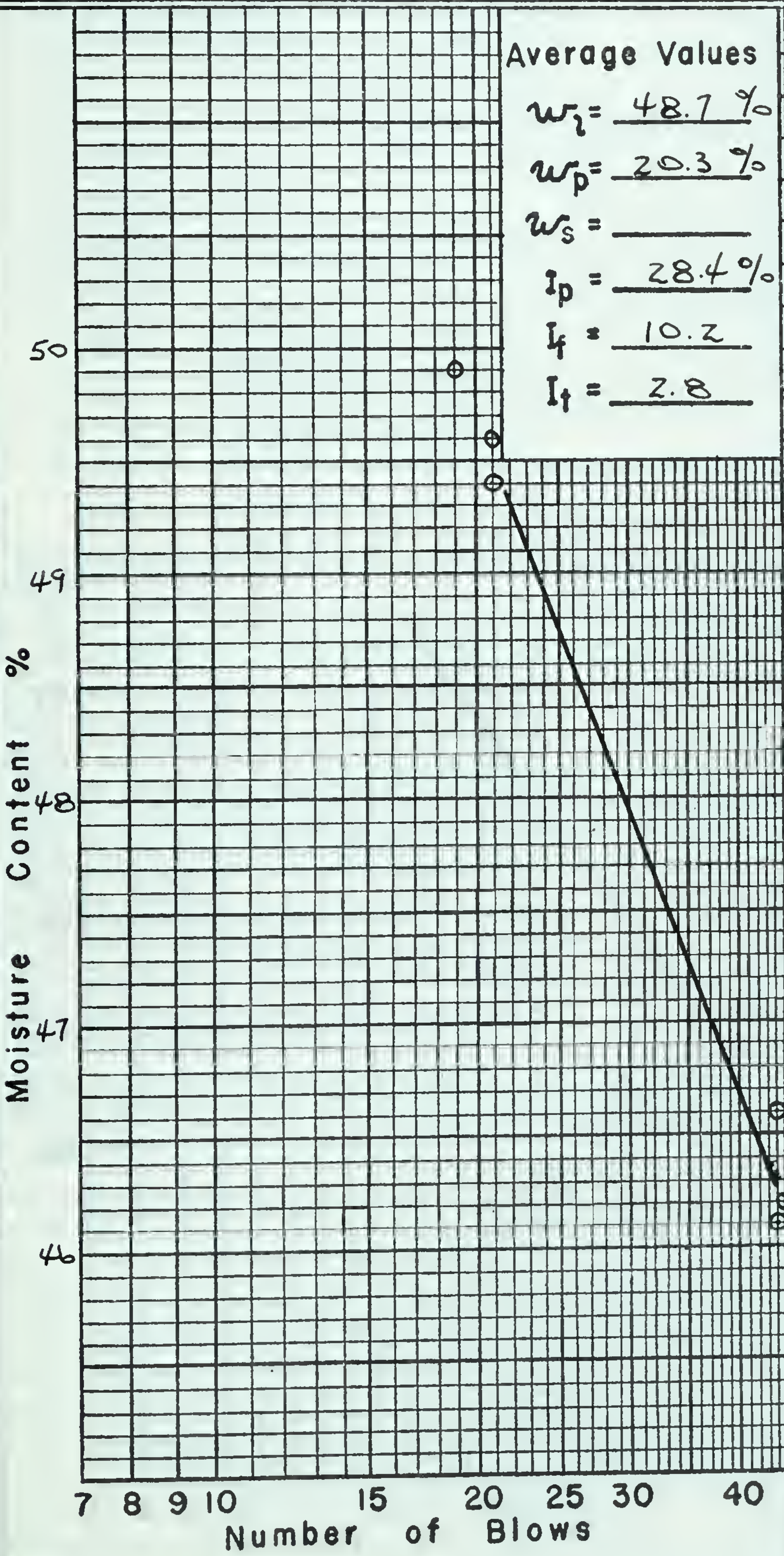
SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

PROJECT Test Pile #4
SITE 114^{th} Ave & 144^{th} Street.
SAMPLE #6
LOCATION
HOLE #4 DEPTH 25'
TECHNICIAN P. K. DATE 1/8/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	44	44	45	21	21	19
Container No.	V50	V70	1	2	A4	A12
Wt. Sample Wet + Tare	98.9870	97.9526	91.0912	91.7674	92.6481	88.0427
Wt. Sample Dry + Tare	93.4766	92.9262	85.6984	86.1965	87.3264	82.4810
Wt. Water	5.5104	5.0264	5.3928	5.5709	5.3217	5.5617
Tare Container	81.5525	82.1483	74.0352	74.9626	76.5604	71.3514
Wt. of Dry Soil	11.9241	10.7779	11.6632	11.2339	10.7660	11.1296
Moisture Content w%	46.1	46.6	46.2	49.6	49.4	49.9



Average Values

w_l = 48.7 %
w_p = 20.3 %
w_s =
I_p = 28.4 %
I_f = 10.2
I_t = 2.8

Plastic Limit

Trial No.	1	2	3
Container No.	1	2	3
Wt. Sample Wet + Tare	51.0806	51.4886	49.3948
Wt. Sample Dry + Tare	50.4181	50.6825	48.4593
Wt. Water	0.6625	0.8061	0.9355
Tare Container	47.1130	46.7891	43.8264
Wt. of Dry Soil	3.3051	3.8934	4.6329
Moisture Content %	20.0	20.7	20.2

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
GLACIAL TILL:- medium plastic, dense, grey in color, coal and pea gravel present, moist.

Remarks:

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SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #4
SITE 114th Ave & 144th Street.
SAMPLE #4
LOCATION
HOLE #4 DEPTH 23'
TECHNICIAN P.K. DATE 1/10/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	39	39	37	15	17	17
Container No.	A12	V70	A4	1	2	V50
Wt. Sample Wet + Tare	92.2882	106.8438	101.8405	96.0438	99.3642	102.1658
Wt. Sample Dry + Tare	88.6530	98.6050	93.4759	88.3855	90.8490	94.9571
Wt. Water	8.6352	8.2388	8.3646	7.6583	8.5152	7.2087
Tare Container	71.3514	82.1483	76.5604	74.0352	74.9626	81.5525
Wt. of Dry Soil	17.3026	16.4567	16.9155	14.3503	15.3864	13.4046
Moisture Content w%	49.8	50.1	49.4	53.4	53.6	53.6

Average Values

w_l = 51.7 %

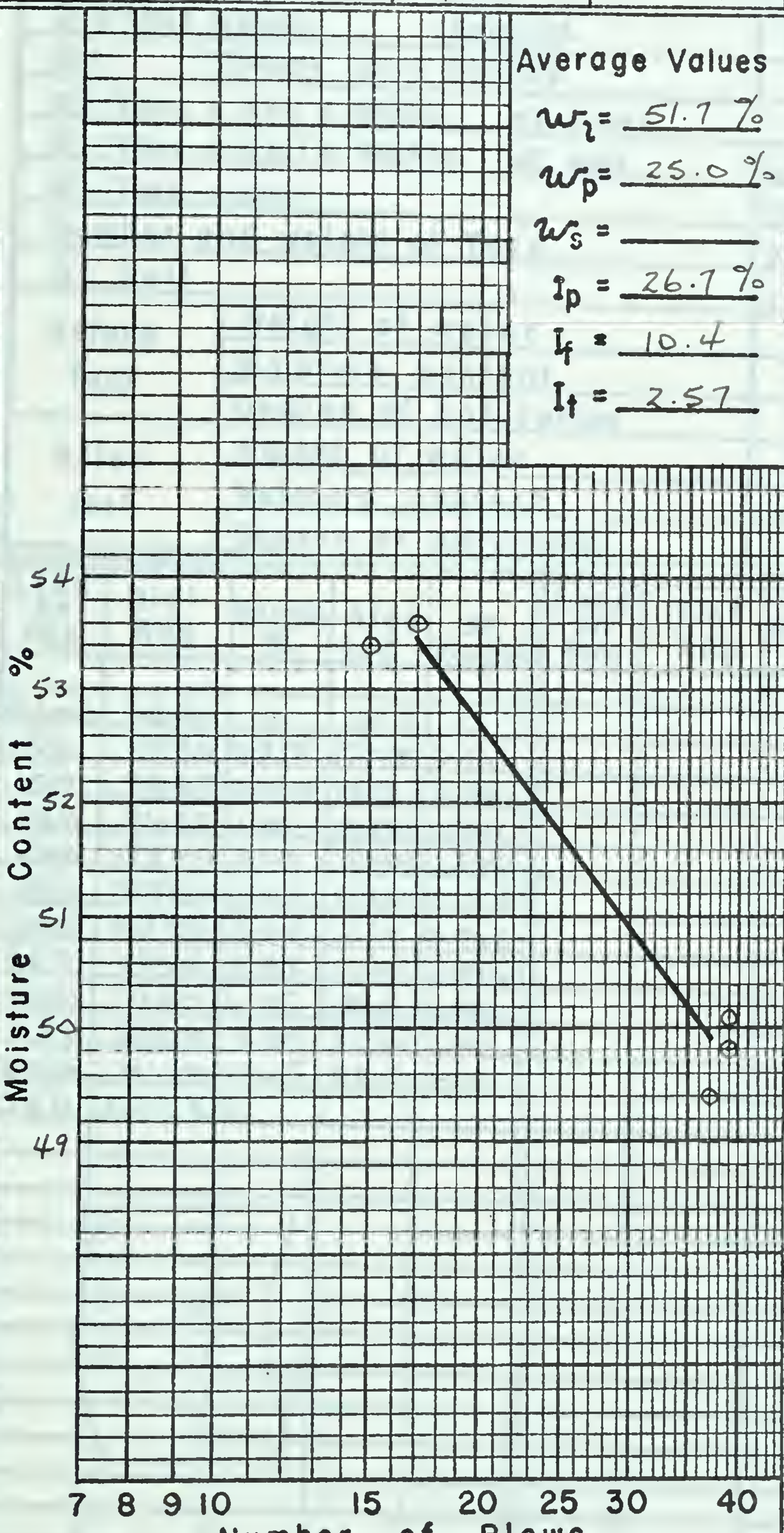
w_p = 25.0 %

w_s = ______

I_p = 26.7 %

I_f = 10.4

I_t = 2.57



Plastic Limit

Trial No.	1	2	3
Container No.	1	2	3
Wt. Sample Wet+Tare	53.2086	52.8943	48.9748
Wt. Sample Dry+Tare	51.9974	51.6783	47.9323
Wt. Water	1.2112	1.2260	1.0425
Tare Container	47.1130	46.7891	43.8264
Wt. of Dry Soil	4.8844	4.8892	4.1059
Moisture Content %	24.8	25.0	25.3

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
GLACIAL TILL:- silty, medium to high plasticity, moist, firm, grey, coal & pea gravel present.

Remarks:

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SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	[illegible]
SITE	[illegible]
SAMPLE	[illegible]
LOCATION	
HOLE	[illegible]
DEPTH	[illegible]
TECHNICIAN	[illegible]
DATE	[illegible]

Liquid Limit

Trial No.	1	2	3	4	5	6
No. of Blows	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Container No.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Wet+Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Dry+Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Moisture Content w%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Plastic Limit

Trial No.	1	2	3
Container No.	[illegible]	[illegible]	[illegible]
Wt. Sample Wet+Tare	[illegible]	[illegible]	[illegible]
Wt. Sample Dry+Tare	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]
Moisture Content %	[illegible]	[illegible]	[illegible]

Average Values

w_L = [illegible]

w_P = [illegible]

w_S =

I_P = [illegible]

I_L = [illegible]

I_F = [illegible]

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_S			

$$w_S = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample

[illegible]

Remarks:

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TRI-AXIAL COMPRESSION

PROJECT Test Pile #4
SITE 114th Ave & 144th Street.
SAMPLE #1
LOCATION
HOLE #4 DEPTH 3'
TECHNICIAN P.K. DATE 1/10/59

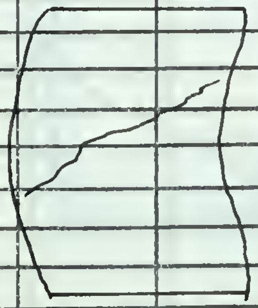
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- highly plastic, moist, firm, nugget structure present.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure (σ_3)							
Length	inches	4.5					
Area Diam. = 1.9"	sq. cms.	18.4					
Volume	c.c.s.	210.0					
Dry Unit Weight	lbs/cu.ft.	82.2					
Gs= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		616.38					
Wt. Tare + Soil		526.54					
Number and weight of Tare		249.96					
Wt. Soil		276.58					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	89.84					
	Moisture content	31.6 %					
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gms	.0240	0	19.0	0										
50	.0090	0.333	19.08	0.131										
100	.9889	0.780	19.15	0.261										
150	.9632	1.35	19.28	0.391										
200	.9300	2.09	19.40	0.515										
250	.8885	3.01	19.60	0.638										
300	.8385	3.58	19.70	0.762										
350	.7755	5.52	20.01	0.875										
400	.7100	6.75	20.4	0.980										
450	.6020	9.39	21.0	1.07										
500	.4100	13.65	22.0	1.14										
550	FAILED.													



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SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #4
SITE 114th Ave & 144th Street.
SAMPLE #2
LOCATION
HOLE #4 DEPTH 8'
TECHNICIAN P.K. DATE 1/10/59

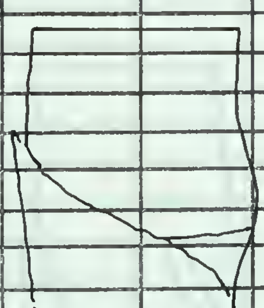
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- highly plastic, moist, firm, nugget structure present.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.7					
Area Diam. = 1.9"		sq. cms.	18.4					
Volume		c.c.s.	219.7					
Dry Unit Weight		lbs/cu.ft.	80.0					
G_S= ·Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			635.38					
Wt. Tare + Soil			531.73					
Number and weight of Tare			249.92					
Wt. Soil			281.81					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		103.65					
	Moisture content		36.8%					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm²	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.0520	0	18.4	0										
50	.0385	0.287	18.47	0.136										
100	.0126	0.837	18.58	0.269										
150	.9840	1.447	18.68	0.402										
200	.9535	2.09	18.79	0.532										
250	.9162	2.88	18.96	0.660										
300	.8750	3.76	19.12	0.784										
350	.8305	4.71	19.35	0.904										
400	.7685	6.01	19.6	1.02										
450	.6835	7.84	20.0	1.12										
500	FAILED													



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SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT Test Pile #4
SITE 114th Ave & 144th Street.
SAMPLE #3
LOCATION
HOLE #4 DEPTH 13'
TECHNICIAN P.K. DATE 1/10/59

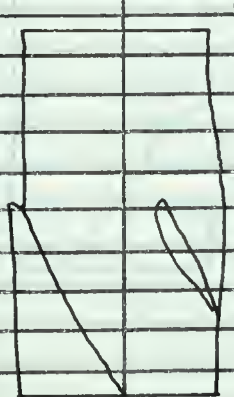
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample:
CLAY:- highly plastic, moist, grey in color.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure (σ_3)							
Length	inches	4.8					
Area Diam. = 1.9"	sq. cms.	18.4					
Volume	c. c. s.	224.0					
Dry Unit Weight	lbs/cu. ft.	77.5					
G_S= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		413.88					
Wt. Tare + Soil		308.82					
Number and weight of Tare		30.74					
Wt. Soil		278.08					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	105.06					
	Moisture content	37.8%					
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.0259	0	18.4	0										
50	.0158	0.210	18.45	0.135										
100	.0000	0.539	18.51	0.27										
150	.9858	0.834	18.58	0.404										
200	.9858	0.834	18.58	0.538										
250	.9430 = .6680 (NEW SETTING)	1.73						18.74	0.667					
300	.6601	1.875	18.77	0.800										
350	.6418	2.27	18.82	0.930										
400	.6190	2.75	18.9	1.06										
450	.5840	3.48	19.1	1.18										
500	FAILED													



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SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #4
SITE 114th Ave & 144th Street.
SAMPLE #4
LOCATION
HOLE #4 DEPTH 18'
TECHNICIAN P.K. DATE 1/19/59

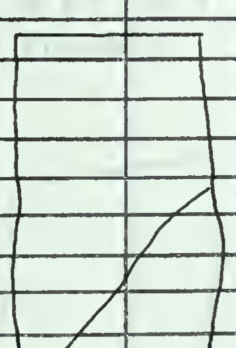
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: CLAY:- highly plastic, stiff, moist, grey in color.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_{3})						
Length		inches	4.8					
Area Diam. = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	200.0					
Dry Unit Weight		lbs/cu.ft.	82.2					
G_S= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			615.37					
Wt. Tare + Soil			513.43					
Number and weight of Tare			249.99					
Wt. Soil			263.44					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		101.94					
	Moisture content		38.7%					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gms	.7262	0	16.4	0										
50	.6809	0.943	16.55	0.151										
100	.6335	1.930	16.72	0.299										
150	.5780	3.09	16.92	0.444										
200	.5192	5.63	17.38	0.575										
250	.4455	5.85	17.4	0.72										
300	.3412	8.04	17.9	0.84										
350	FAILED													



UNIVERSITY of ALBERTA DEPT. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **TRI-AXIAL COMPRESSION**	
	PROJECT Test Pile # 4
	SITE 114th Ave & 144th Street.
	SAMPLE # 5
	LOCATION
	HOLE # 4 DEPTH 23'
	TECHNICIAN P.K. DATE 1/12/59

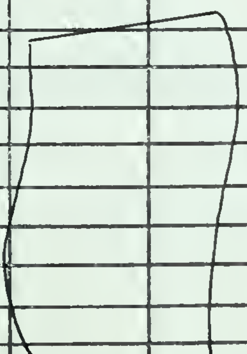
Machine Data:- Unconfined Test.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: Glacial Till:- silty, medium to high plasticity, moist, firm, grey, coal & pea gravel present.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure (σ_3)							
Length	inches	4.3					
Area Diam. = 1.85"	sq. cms.	17.4					
Volume	c. c. s.	197.5					
Dry Unit Weight	lbs/cu. ft.	110.0					
G_S= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		440.91					
Wt. Tare + Soil		378.51					
Number and weight of Tare		30.69					
Wt. Soil		347.82					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	62.40					
	Moisture content	18.0%					
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0	.9965	0	17.4	0										
50	.9735	0.534	17.51	0.143										
100	.9542	0.981	17.58	0.321										
150	.9288	1.571	17.70	0.424										
200	.8865	2.555	17.89	0.559										
250	.8431	3.56	18.08	0.692										
300	.7840	4.94	18.31	0.820										
350	.6970	6.95	18.70	0.936										
400	.5060	11.40	19.65	1.02										
450	.4060	13.75	20.2	1.12										
500	.3132	15.9	20.7	1.21										
550	FAILED.													



UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT Test Pile #4
SITE
SAMPLE #6
LOCATION
HOLE #4 DEPTH 25'.
TECHNICIAN P.K. DATE 2/1/59

Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: GLACIAL TILL:- dense, grey, medium plastic, sandy, coal & pea gravel present.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	3.8					
Area Diam. = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	158.2					
Dry Unit Weight		lbs/cu.ft.	110.0					
Gs= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			380.17					
Wt. Tare + Soil			327.33					
Number and weight of Tare			48.13					
Wt. Soil			279.20					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		52.84					
	Moisture content		18.9 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.8972													
50	.8752													
100	.8095	BAD TEST												
150	.7250													
200	.6329													
250	.5130													
300	FAILED.													

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SOIL MECHANICS LABORATORY

MOISTURE CONTENT

PROJECT Test Pile #4.
SITE 114th Ave & 144th Street.
SAMPLE
LOCATION
HOLE #4 DEPTH
TECHNICIAN P.K. DATE 1/2/59

Hole No·	4	4	4	4	4	4
Depth	3'	5'	8'	10'	13'	15'
Sample No·	1		2		3	
Container No·	1A27	1A23	1A27	1A35	1A86	1A53
Wt· Sample Wet + Tare	54.48	81.61	63.27	89.44	56.91	82.27
Wt· Sample Dry + Tare	44.79	65.58	50.81	69.54	46.01	64.88
Wt· Water	9.69	16.03	12.46	19.90	10.90	17.39
Tare Container	17.67	17.27	17.67	17.71	17.38	17.55
Wt· of Dry Soil	27.12	48.31	33.14	51.83	28.63	47.33
Moisture Content w %	35.7	33.2	37.6	38.5	38.1	36.8
Hole No	4	4	4	4		
Depth	18'	20'	23'	25'		
Sample No·	4		5	6		
Container No·	1A27	1A17	1A67	1A86		
Wt· Sample Wet + Tare	56.28	96.35	103.80	55.35		
Wt· Sample Dry + Tare	45.91	75.20	90.07	49.66		
Wt· Water	10.37	21.15	13.73	5.69		
Tare Container	17.67	17.43	40.61	17.38		
Wt· of Dry Soil	28.24	57.77	49.46	32.28		
Moisture Content w %	36.4	36.6	27.8	17.6		
Hole No·						
Depth						
Sample No·						
Container No·						
Wt· Sample Wet + Tare						
Wt· Sample Dry + Tare						
Wt· Water						
Tare Container						
Wt· of Dry Soil						
Moisture Content w%						

Remarks:

LABORATORY TEST RESULTS

TEST PILE NO. 5

UNIVERSITY of ALBERTA
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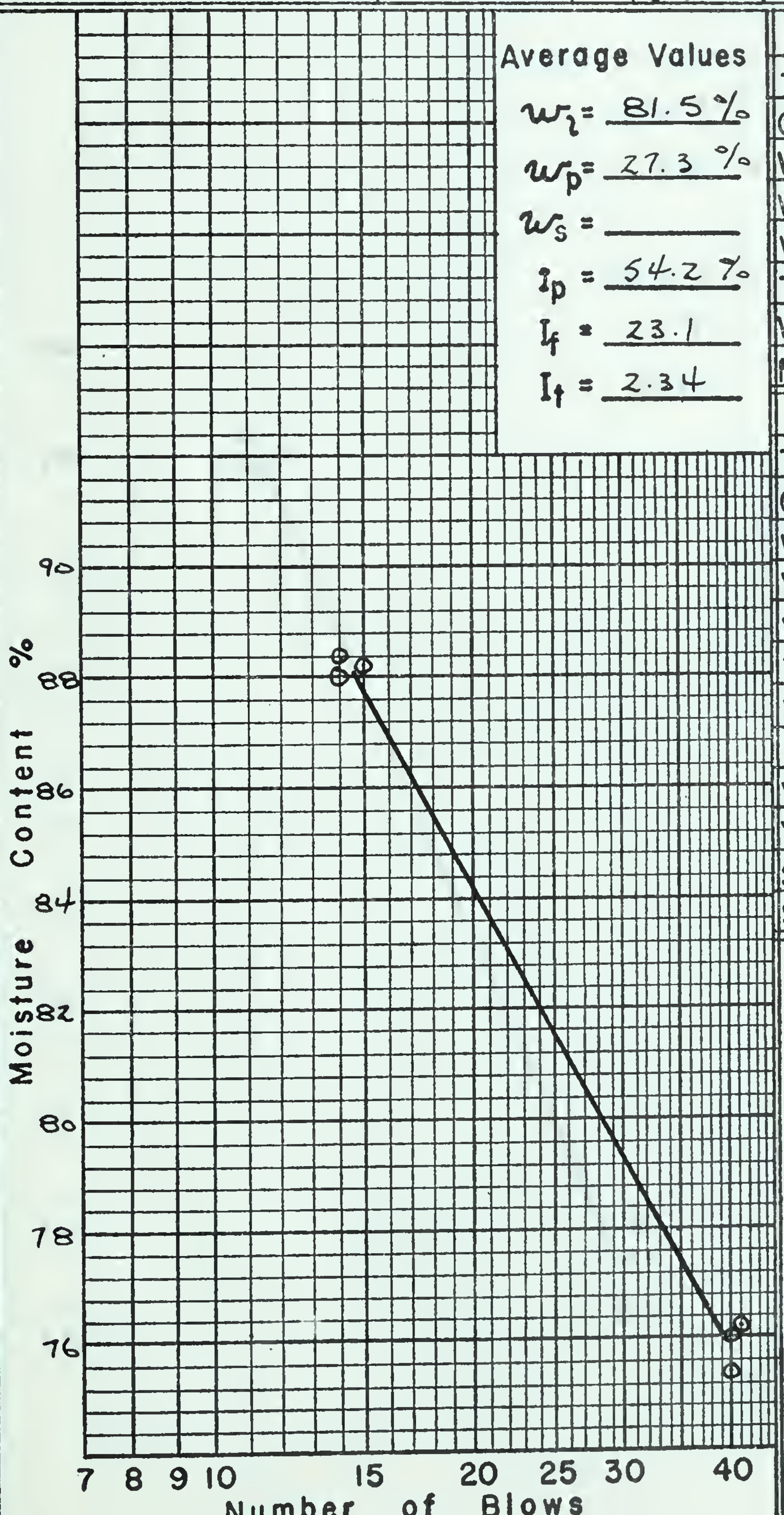
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #5
SITE 114th Ave & 144th Street
SAMPLE #1
LOCATION
HOLE #5 DEPTH 4'
TECHNICIAN P.K. DATE 1/7/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	40	41	40	15	14	14
Container No.	V36	V41	V46	V79	V20	V71
Wt. Sample Wet + Tare	82.0120	84.3073	99.4312	93.5888	79.4790	86.2867
Wt. Sample Dry + Tare	75.5530	77.9705	92.6951	86.9911	72.9527	79.6222
Wt. Water	6.4590	6.3368	6.7361	6.5977	6.5263	6.6645
Tare Container	67.0527	69.6448	83.7237	79.5047	65.5706	72.0433
Wt. of Dry Soil	8.5003	8.3257	8.9714	7.4864	7.3821	7.5789
Moisture Content w%	76.0	76.2	75.3	88.2	88.4	88.0



Average Values

w_l = 81.5 %
w_p = 27.3 %
w_s =
I_p = 54.2 %
I_f = 23.1
I_t = 2.34

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	32.7891	33.2766	42.3390
Wt. Sample Dry + Tare	32.1599	32.5604	41.6505
Wt. Water	0.6292	0.7162	0.6885
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	2.2696	2.6590	2.5443
Moisture Content %	27.7	27.0	27.1

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
Clay:- highly plastic, firm, nugget structure present, grey, moist.

Remarks:

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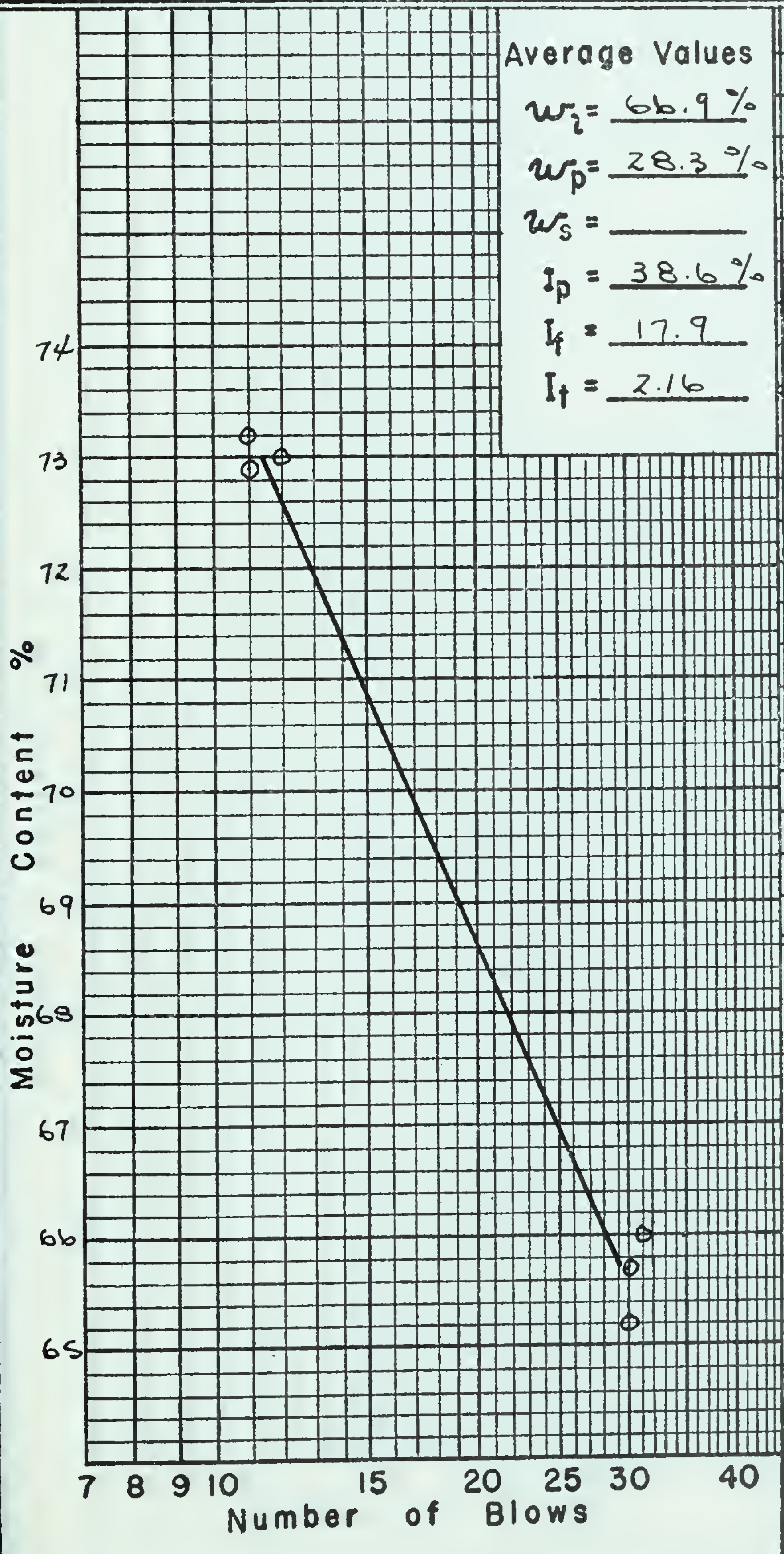
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	TEST PILE #5
SITE	114th AVE & 144th STREET.
SAMPLE	#2
LOCATION	
HOLE	#5 — DEPTH 9'
TECHNICIAN	P.K. — DATE 11/6/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	31	30	30	11	11	12
Container No.	1	V50	A4	2	V70	A12
Wt. Sample Wet + Tare	88.2527	94.5766	91.0518	87.2670	98.7496	90.9610
Wt. Sample Dry + Tare	82.5995	89.4086	85.3197	82.0701	91.7564	82.6886
Wt. Water	5.6532	5.1680	5.7321	5.1969	6.9932	8.2724
Tare Container	74.0352	81.5525	76.5604	74.9626	82.1483	71.3514
Wt. of Dry Soil	8.5643	7.8561	8.7593	7.1075	9.6081	11.3372
Moisture Content w%	66.0	65.7	65.2	73.2	72.9	73.0



Average Values

w_l = 66.9 %

w_p = 28.3 %

w_s =

I_p = 38.6 %

I_f = 17.9

I_t = 2.16

Plastic Limit

Trial No.	1	2	3
Container No.	1	2	3
Wt. Sample Wet + Tare	49.9075	50.5913	46.8976
Wt. Sample Dry + Tare	49.2945	49.7550	46.2114
Wt. Water	0.6130	0.8363	0.6862
Tare Container	47.1130	46.7891	43.8264
Wt. of Dry Soil	2.1815	2.9659	2.3850
Moisture Content %	28.1	28.2	28.7

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: CLAY:- highly plastic, moist firm, brownish grey in color, nugget structure present.

Remarks:

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SOIL MECHANICS LABORATORY

ATTERBERG LIMITS

PROJECT [illegible]
SITE [illegible]
SAMPLE [illegible]
LOCATION
HOLE [illegible] DEPTH [illegible]
TECHNICIAN [illegible] DATE [illegible]

Liquid Limit

Trial No.	1	2	3	4	5	6
No. of Blows	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Container No.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Wet + Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Dry + Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Moisture Content w%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Plastic Limit

Trial No.	1	2	3
Container No.	[illegible]	[illegible]	[illegible]
Wt. Sample Wet + Tare	[illegible]	[illegible]	[illegible]
Wt. Sample Dry + Tare	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]
Moisture Content %	[illegible]	[illegible]	[illegible]

Average Values

w_l = [illegible]

w_p = [illegible]

w_s =

I_p = [illegible]

I_f = [illegible]

I_t = [illegible]

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_s			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_s			
Shrinkage Vol. $V - V_s$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V - V_s}{W_s} \times 100 \right)$$

Description of Sample:

[illegible]

Remarks:

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SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	TEST PILE #5		
SITE	114th AVE & 144th STREET.		
SAMPLE	#3		
LOCATION			
HOLE	#5	DEPTH	14'
TECHNICIAN	P.K.	DATE	1/12/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	29	27	28	13	13	13
Container No.	V20	V36	V71	V79	V41	V46
Wt. Sample Wet + Tare	88.1863	87.3462	90.2118	102.9365	94.3058	106.7643
Wt. Sample Dry + Tare	79.2067	79.3354	83.0734	93.2599	84.2027	97.2281
Wt. Water	8.9796	8.0108	7.1384	9.6766	10.1031	9.5362
Tare Container	65.5706	67.0527	72.0433	79.5047	69.6448	83.7237
Wt. of Dry Soil	13.6361	12.2827	11.0301	13.7552	14.5579	13.5044
Moisture Content w%	65.8	65.4	64.9	70.4	69.6	70.6

Average Values

w_l = 66.0 %

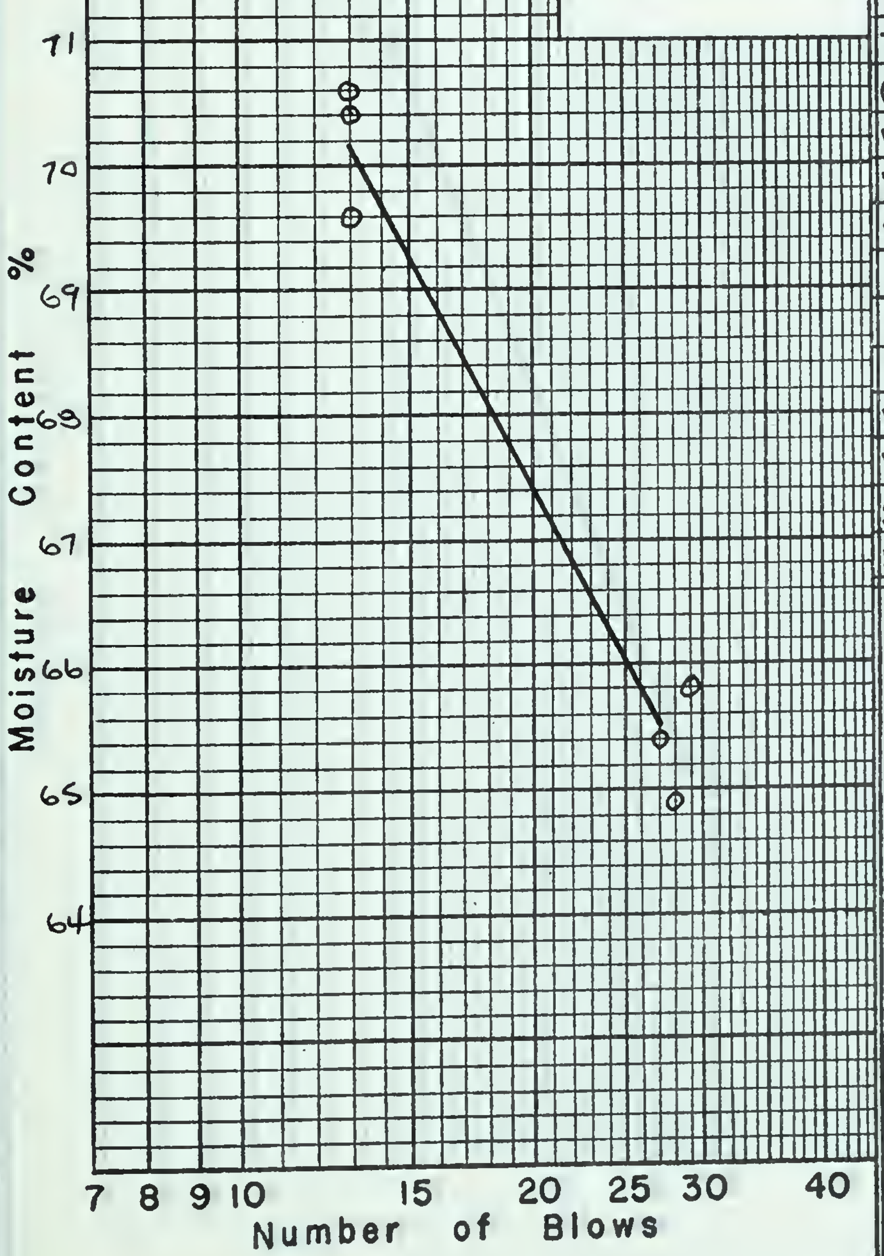
w_p = 29.6 %

w_s = ______

I_p = 36.4 %

I_f = 14.7

I_t = 2.48



Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet+Tare	32.9343	32.8739	41.8710
Wt. Sample Dry+Tare	32.2399	32.1970	41.2407
Wt. Water	0.6944	0.6769	0.6303
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	2.3496	2.2956	2.1345
Moisture Content %	29.6	29.5	29.6

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: CLAY:- highly plastic, grey, firm, moist.

Remarks:

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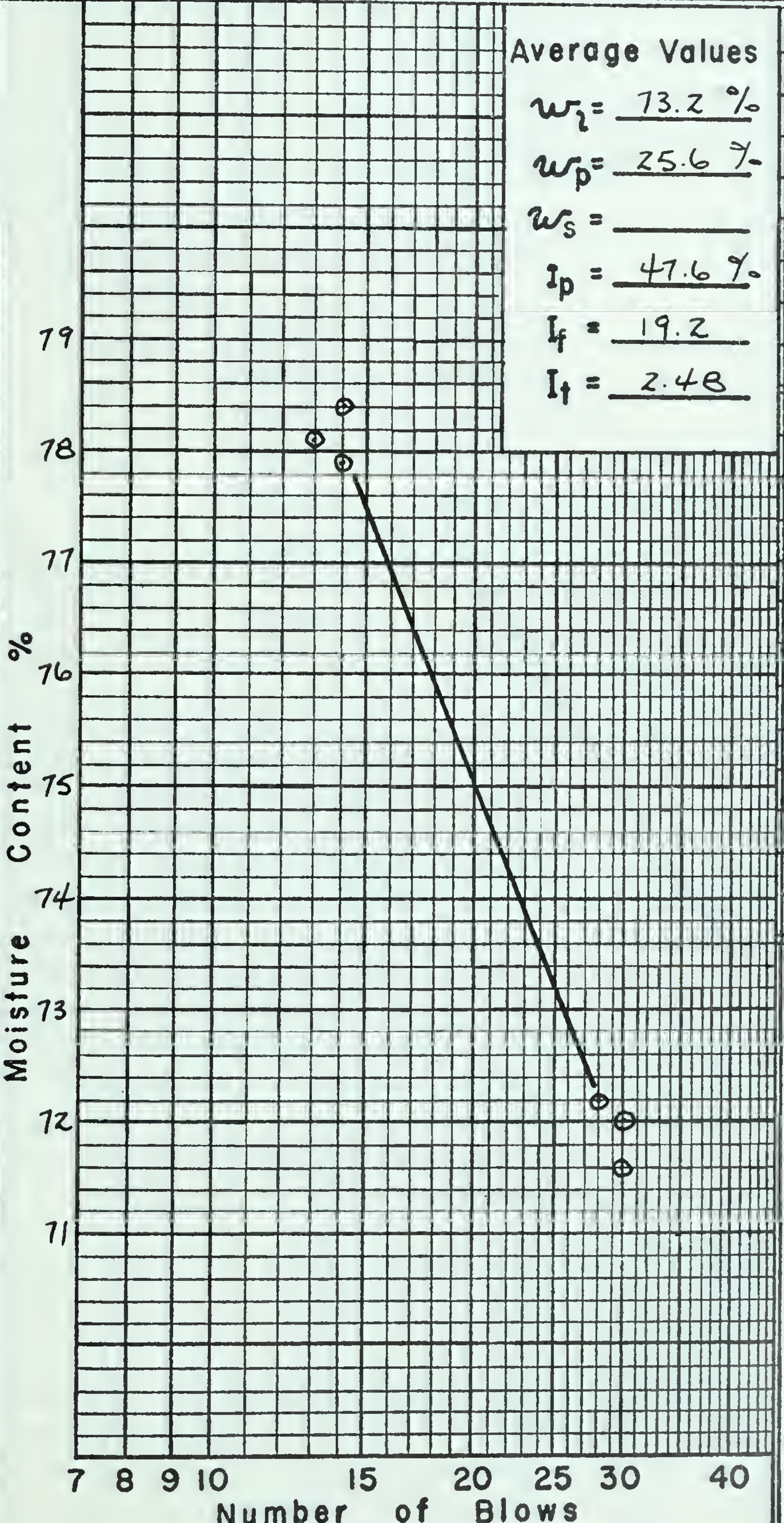
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT Test Pile #5
SITE 114th Ave & 144th Street.
SAMPLE #4
LOCATION
HOLE #5 DEPTH 19'
TECHNICIAN P.K. DATE 1/20/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	30	30	28	14	13	14
Container No.	V20	V36	V41	V46	V79	V71
Wt. Sample Wet + Tare	87.7516	91.9382	93.5175	109.3431	99.5526	98.6826
Wt. Sample Dry + Tare	78.4710	81.5530	83.5179	98.0745	90.7679	87.0067
Wt. Water	9.2806	10.3852	9.9996	11.2686	8.7847	11.6759
Tare Container	65.5706	67.0527	69.6448	83.7237	79.5047	72.0433
Wt. of Dry Soil	12.9004	14.5003	13.8731	14.3508	11.2632	14.9634
Moisture Content w%	72.0	71.6	72.2	78.4	78.1	77.9



Average Values

w_l = 73.2 %
w_p = 25.6 %
w_s = ______
I_p = 47.6 %
I_f = 19.2
I_t = 2.48

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	33.2088	32.4240	41.6409
Wt. Sample Dry + Tare	32.5333	31.9171	41.1200
Wt. Water	0.6755	0.5069	0.5209
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	2.6430	2.0157	2.0138
Moisture Content %	25.6	25.2	25.9

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample:
CLAY:- highly plastic, firm, moist, grey in color.

Remarks:

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SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	[illegible]
SITE	[illegible]
SAMPLE	[illegible]
LOCATION	
HOLE	[illegible]
DEPTH	[illegible]
TECHNICIAN	[illegible]
DATE	[illegible]

Liquid Limit

Trial No.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
No. of Blows	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Container No	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Wet + Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Sample Dry + Tare	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Moisture Content w%	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Plastic Limit

Trial No.	1	2	3
Container No.	[illegible]	[illegible]	[illegible]
Wt. Sample Wet + Tare	[illegible]	[illegible]	[illegible]
Wt. Sample Dry + Tare	[illegible]	[illegible]	[illegible]
Wt. Water	[illegible]	[illegible]	[illegible]
Tare Container	[illegible]	[illegible]	[illegible]
Wt. of Dry Soil	[illegible]	[illegible]	[illegible]
Moisture Content %	[illegible]	[illegible]	[illegible]

Average Values

w_L = [illegible]

w_P = [illegible]

w_S =

I_P = [illegible]

I_L = [illegible]

I_F = [illegible]

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_s			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_0			
Shrinkage Vol. $V-V_0$			
Shrinkage Limit w_S			

$$w_S = w - \left(\frac{V - V_0}{W_s} \times 100\right)$$

Description of Sample: [illegible]

Remarks:

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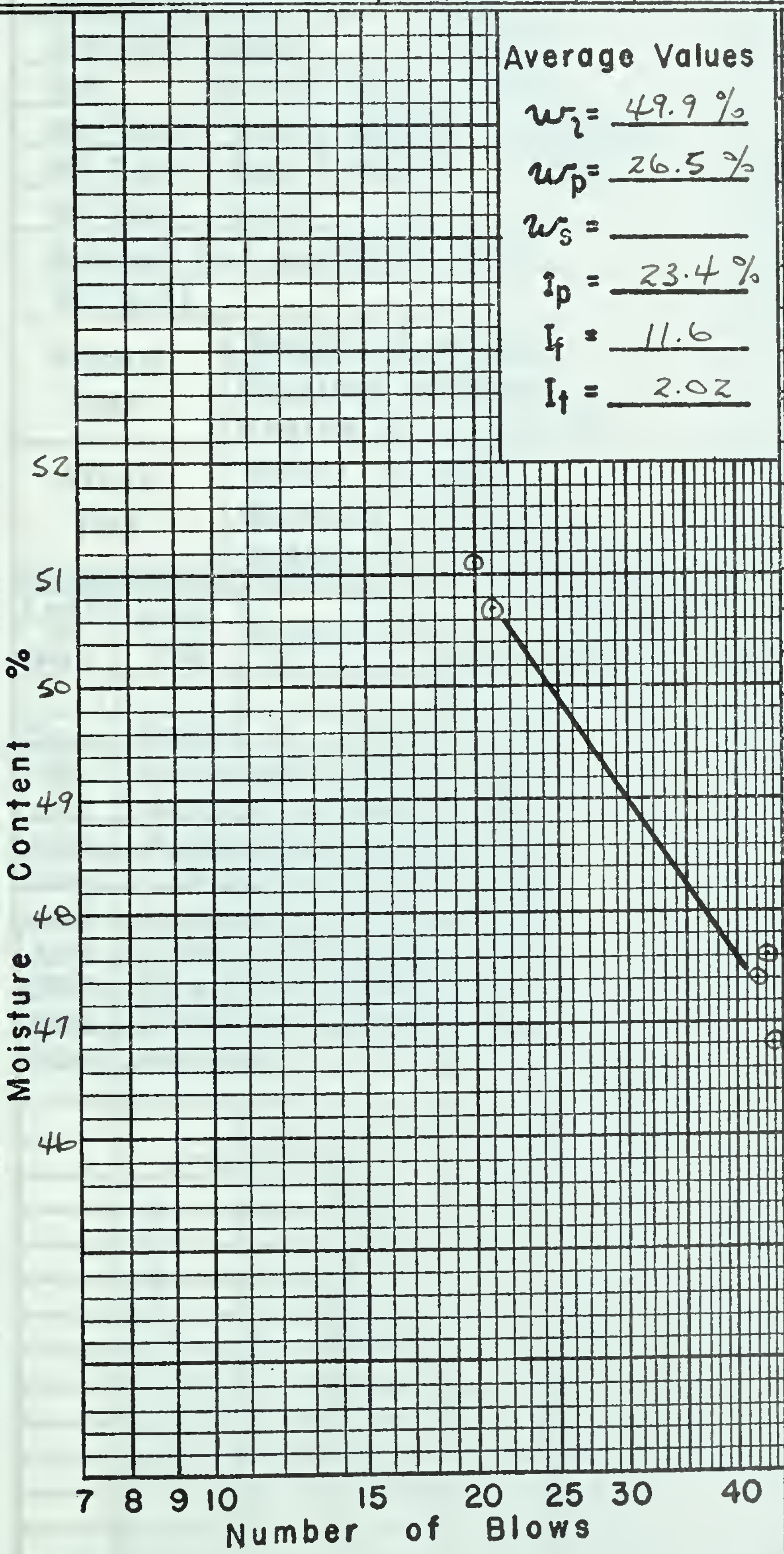
SOIL MECHANICS LABORATORY
ATTERBERG LIMITS

PROJECT	Test Pile #5		
SITE			
SAMPLE	#5		
LOCATION			
HOLE	#5	DEPTH	24'
TECHNICIAN	P.K.	DATE	1/30/59

Liquid Limit

Trial No.	1	2	3	1	2	3
No. of Blows	43	42	44	20	20	21
Container No.	V79	V36	V71	V41	V46	V20
Wt. Sample Wet + Tare	106.7945	97.3334	101.9460	96.2995	110.9502	96.1273
Wt. Sample Dry + Tare	97.9983	87.6125	92.4382	87.2984	101.7576	85.8403
Wt. Water	8.7962	9.7209	9.5078	9.0011	9.1926	10.2870
Tare Container	79.5047	67.0527	72.0433	69.6448	83.7237	65.5706
Wt. of Dry Soil	18.4936	20.5598	20.3949	17.6536	18.0339	20.2697
Moisture Content w%	47.6	47.4	46.8	51.1	51.1	50.7



Average Values

w_l = 49.9 %
w_p = 26.5 %
w_s =
I_p = 23.4 %
I_f = 11.6
I_t = 2.02

Plastic Limit

Trial No.	1	2	3
Container No.	4	7	11
Wt. Sample Wet + Tare	33.9745	33.6356	41.9871
Wt. Sample Dry + Tare	33.1157	32.8523	41.3887
Wt. Water	0.8588	0.7833	0.5984
Tare Container	29.8903	29.9014	39.1062
Wt. of Dry Soil	3.2254	2.9509	2.2825
Moisture Content %	26.6	26.5	26.3

Shrinkage Limit

Trial No.			
Container No.			
Wt. Sample Wet + Tare			
Wt. Sample Dry + Tare			
Wt. Water			
Tare Container			
Wt. of Dry Soil W_o			
Moisture Content w%			
Vol. Container V			
Vol. Dry Soil Pat V_o			
Shrinkage Vol. $V-V_o$			
Shrinkage Limit w_s			

$$w_s = w - \left(\frac{V-V_o}{W_o} \times 100\right)$$

Description of Sample: GLACIAL TILL:- silty, medium to high plasticity, moist, firm, grey in color, coal & pea gravel present.

Remarks:

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #5
SITE 114th Ave & 144th Street.
SAMPLE #1
LOCATION
HOLE #5 DEPTH 4'
TECHNICIAN P.K. DATE 1/2/59

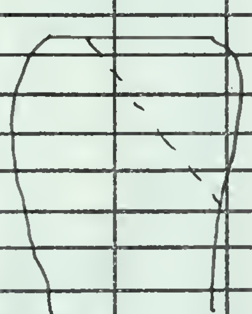
Machine Data:- Unconfined Test.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: Clay:- highly plastic firm, nugget structure present, grey, moist.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure (σ_3)							
Length	inches	4.1					
Area Diam. = 1.8"	sq. cms.	16.4					
Volume	c.c.s.	170.8					
Dry Unit Weight	lbs/cu.ft.	86.7					
G_S= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		342.07					
Wt. Tare + Soil		268.09					
Number and weight of Tare		30.75					
Wt. Soil		237.34					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	73.98					
	Moisture content	31.2 %					
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gm	.9092	0	16.4	0										
50	.8930	0.395	16.47	0.152										
100	.8700	0.713	16.52	0.303										
150	.8451	1.56	16.68	0.45										
200	.7059	4.96	17.09	0.586										
250	.6621	6.02	17.48	0.715										
300	.6032	7.46	17.72	0.847										
350	.5230	9.43	18.1	0.97										
400	.3915	12.61	18.8	1.06										
450	FAILED													

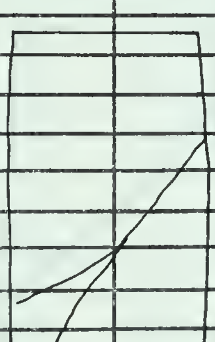


UNIVERSITY of ALBERTA DEPT. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **TRI-AXIAL COMPRESSION**	PROJECT Test Pile #5 SITE 114th Ave & 144th Street SAMPLE #2 LOCATION HOLE #5 DEPTH 9' TECHNICIAN D.K. DATE 1/15/59
Machine Data:- UNCONFINED TEST. Machine No. 9 Multiplication Factor x 50 Wt. Loading Block + Piston (gms.)	Description of Sample: CLAY:- highly plastic, moist, firm, brownish grey in color, nugget structure present.

SPECIMEN DATA

Specimen Number		1	2	3	4	5	6
Lateral Pressure (σ_3)							
Length	inches	4.2					
Area Diam. = 1.9"	sq. cms.	18.4					
Volume	c.c.s.	196.0					
Dry Unit Weight	lbs/cu.ft.	78.5					
G_S= · Volume Soil Solids							
Wt. Tare + Soil + Water at start							
Wt. Tare + Soil + Water at end		585.14					
Wt. Tare + Soil		496.67					
Number and weight of Tare		249.93					
Wt. Soil		246.74					
Before Test	Weight of water						
	Moisture content						
	Degree of saturation						
After Test	Weight of water	88.47					
	Moisture content	35.8%					
	Degree of saturation						

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.8642	0	18.4	0										
50	.8468	0.414	18.48	0.135										
100	.8118	1.249	18.64	0.268										
150	.7781	2.05	18.80	0.399										
200	.7430	2.884	18.96	0.527										
250	.7092	3.69	19.12	0.654										
300	.6760	4.48	19.26	0.78										
350	.6400	5.34	19.45	0.90										
400	.5980	6.34	19.68	1.02										
450	FAILED													



UNIVERSITY of ALBERTA
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SOIL MECHANICS LABORATORY

TRI-AXIAL COMPRESSION

PROJECT Test Pile #5
SITE 114th Ave & 144th Street.
SAMPLE #3
LOCATION
HOLE #5 DEPTH 14'
TECHNICIAN P.K. DATE 1/10/59

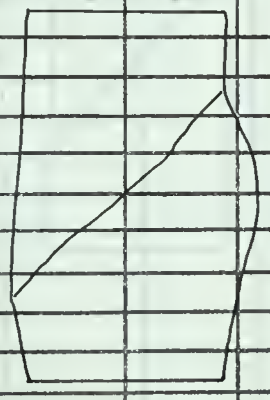
Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample:
CLAY:- highly plastic, grey, firm, moist.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.3					
Area Diam. = 1.8"		sq. cms.	16.4					
Volume		c.c.s.	179.0					
Dry Unit Weight		lbs/cu.ft.	88.6					
G_S= · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			594.52					
Wt. Tare + Soil			504.31					
Number and weight of Tare			249.96					
Wt. Soil			254.35					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		90.21					
	Moisture content		35.4%					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm²	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0gm	.8770	0	16.4	0										
50	.8570	0.465	16.48	0.152										
100	.8365	0.941	16.57	0.302										
150	.8125	1.50	16.65	0.451										
200	.7869	2.09	16.74	0.597										
250	.7593	2.73	16.88	0.741										
300	.7310	3.39	16.98	0.884										
350	.6955	4.22	17.13	1.02										
400	.6491	5.30	17.32	1.15										
450	.5735	7.06	17.70	1.27										
500	FAILED													



UNIVERSITY of ALBERTA DEPT. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **TRI-AXIAL COMPRESSION**	PROJECT Test Pile #5 SITE 114th Ave & 144th Street. SAMPLE #4 LOCATION HOLE #5 DEPTH 19' TECHNICIAN P.K. DATE 1/2/59

Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms)

Description of Sample: CLAY:- highly plastic, firm, moist, grey in color.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.0					
Area	Diam. = 1.8"	sq. cms.	16.4					
Volume		c. c. s.	166.5					
Dry Unit Weight		lbs/cu. ft.	82.8					
Gs =		Volume Soil Solids						
Wt. Tare + Soil + Water		at start						
Wt. Tare + Soil + Water		at end	337.97					
Wt. Tare + Soil			251.72					
Number and weight of Tare			30.76					
Wt. Soil			220.96					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		86.25					
	Moisture content		39.0 %					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gm	.5091	0	16.4	0										
50	.5045	0.115	16.43	0.152										
100	.4635	1.14	16.60	0.301										
150	.3612	3.70	17.02	0.441										
200	.2490	6.51	17.55	0.57										
250	.0658	11.06	18.45	0.68										
300	FAILED													

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
TRI-AXIAL COMPRESSION

PROJECT Test Pile #5
SITE 114th Ave & 144th Street.
SAMPLE #5
LOCATION
HOLE #5 DEPTH 24'
TECHNICIAN P.K. DATE 1/10/59

Machine Data:- UNCONFINED TEST.
Machine No. 9
Multiplication Factor x 50
Wt. Loading Block + Piston (gms.)

Description of Sample: GLACIAL TILL:- silty, medium to high plasticity, moist, firm, grey coal & pea gravel present.

SPECIMEN DATA

Specimen Number			1	2	3	4	5	6
Lateral Pressure		(σ_3)						
Length		inches	4.4					
Area Diam. = 1.85"		sq. cms.	17.4					
Volume		c.c.s.	208.0					
Dry Unit Weight		lbs/cu.ft.	108.0					
Gs = · Volume Soil Solids								
Wt. Tare + Soil + Water at start								
Wt. Tare + Soil + Water at end			443.44					
Wt. Tare + Soil			379.41					
Number and weight of Tare			30.68					
Wt. Soil			348.73					
Before Test	Weight of water							
	Moisture content							
	Degree of saturation							
After Test	Weight of water		64.03					
	Moisture content		18.4%					
	Degree of saturation							

Load on Pan	Dial Rdg	Strain %	Area cm^2	σ_1 Kgm/cm^2	Load on Pan	Dial Rdg.	Strain	Area	σ_1	Load on Pan	Dial Rdg.	Strain	Area	σ_1
0 gm	.9387	0	17.4	0										
50	.9215	0.391	17.48	0.143										
100	.9090	0.675	17.53	0.285										
150	.8800	1.334	17.64	0.425										
200	.8461	2.102	17.79	0.562										
250	.8131	2.85	17.91	0.697										
300	.7785	3.64	18.06	0.831										
350	.7431	4.44	18.22	0.961										
400	.6872	5.72	18.48	1.082										
450	.6278	7.06	18.72	1.202										
500	.5650	8.48	19.0	1.32										
550	.4450	11.20	19.6	1.40										
600	FAILED